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THE ABOLITION OF GRADE CROSSINGS in Massachusetts is treated at considerable length in the late message of the Governor of that State. The general law of 1890, defining the policy of the State as to the abolition of grade crossings, has practically ceased to be operative. The $5,000,000 appropriated by that act has been nearly expended, the sum actually spend by the State being $4,749,736, and the railway companies have paid out $12,976,017 under this act. Under special acts by the Legislature the State has expended $2,728,084, and the railways $6,062,409, in addition to the above amounts; or totals of $7,477,820 and $19,038,427 respectively. The Governor recommends an appropriation of $5,000,000 more to be expended in the further operation of the general law, to be paid out in sums not exceeding $550,000 in any one year. The railroad commissioners are to decide what grade crossings should be abolished. The Governor believes that street railways should contribute to the expense of removing grade crossings, when their lines use the crossings; and they should also be allowed to petition for such abolition. This is in accord with the report of the railroad commissioners who were ordered to examine into this matter by the Legislature of 1900. It is also recommended that the amount contributed by the street railway companies be deducted from the 25% otherwise paid by the commonwealth.

THE NEW TERMINAL STATION at Van Buren St. and Pacific Ave., Chicago, for the Lake Shore & Michigan Southern Ry. and Chicago, Rock Island & Pacific Ry. is to be built during the present year. The old station has been abandoned, and will at once be torn down, through trains now using the Grand Central Station, at Harrison St. and Fifth Ave., while suburban trains will use a temporary station at Harrison St., just south of the old train shed. The general plans of the new station have been prepared, but the details have not yet been worked out. The architects are Frost & Granger, of Chicago.

NEW RAILWAY CONSTRUCTION in the United States during the year 1901 aggregated over 5,000 miles and exceeded that of any year since 1890. The "Railway Age" and "Railroad Gazette," both of which publish statements at the end of the year, give the mileage of new road as 5,057 and 4,518 miles, respectively. These figures are very approximate, the former showing an increase and the latter a decrease as compared with the previous year. The figures would have been still greater but for the difficulty in obtaining rails, as several hundred miles of road have been graded and made ready for the track. Five eastern states and one western state (Nebraska) have no new road, but in 43 states and territories 332 lines have built 5,057 miles. The highest records are for Texas, 583 miles, and Oklahoma Territory, 428 miles. The largest single line was the extension of the Chicago, Rock Island & Pacific Ry., from Liberal, Kan., to Santa Rosa, N. M., 265 miles. The "Gazette" reports 658 miles of railway built by 163 companies in Canada, while the "Age" reports 362 miles built by 13 lines in Canada; and 439 miles built by 12 lines in Mexico. Its record for the past 12 years is as follows:

1890	5,670 miles.	1896	1,848 miles.
1891	4,281 "	1897	1,880 "
1892	4,192 "	1898	3,083 "
1893	2,635 "	1899	4,588 "
1894	1,949 "	1900	4,437 "
1895	1,803 "	1901	5,057 "

THE NEW YORK RAPID TRANSIT RAILWAY extension to Brooklyn is held to be necessary and for the public interest by the special Supreme Court Commission appointed to report upon this question. This decision means that the Rapid Transit Commission may proceed with the preparations for constructing the proposed extension as soon as the Court formally approves the findings of its commissioners. The report says in part: "Concerning the route proposed from the City Hall in the Borough of Manhattan, to the Battery, this, it is to be observed, is a part of the rapid transit system as originally contemplated, and received as a part of that system the approval of the public authorities. Its construction as a part thereof was, however, abandoned for the time being, in view of the necessity for devising a more economical system than at first proposed. It needs, nevertheless, but a glance at the plans to demonstrate that it is an integral part of the rapid transit system under construction in the Borough of Manhattan, and essential to afford the same conveniences to those transacting business below the City Hall as are afforded to those who transact business above that point. As to the question of cost of the road, it remains for us to direct attention to the fact that it is safely within the statutory limit. Before proceeding to devise and lay out plans for the proposed road, the Board of Rapid Transit Commissioners applied to the municipal authorities ot ascertain what amount would be at their disposal for the construction of the proposed road. They were informed that the amount was from $8,000,000 to $10,000,000. They have proceeded upon this basis, and while it has been suggested in the testimony and stated emphatically by counsel for the board that in bidding for the privilege of constructing and operating the road it will be found that the road will be constructed for much less than $8,000,000, yet it should undoubtedly be assumed that the cost will be about $10,000,000. There remains from the amount authorized to be expended by the New York Rapid Transit act a sum in excess of $10,000,000, but, in view of the financial condition of the city of New York and the questions relating thereto, it is, in our opinion, wise and prudent on the part of the Board of Rapid Transit Railroad Commissioners to provide for the construction of a railroad which will not cost to exceed $10,000,000."

THE MOST SERIOUS RAILWAY ACCIDENT of the week was a rear collision in the Park Ave. tunnel of the New York Central & Hudson River R. R. near 56th St., New York city. The collision occurred at 8.20 a. m., Jan. 8, between a New York, New Haven & Hartford train from Danbury and a train on the New York & Harlem R. R. from White Plains. The first-named train was standing on one of the tracks in the center tunnel when the Harlem locomotive crashed into its rear and telescoped a large part of the rear car, while the two cars next in front were badly wrecked. As we go to press the number killed in the accident is reported from 12 to 20, while 25 or more received serious injuries. The wreck took fire, probably from coals from the locomotive. Trains are run through the tunnel on the absolute block system, and the cause of the collison has not yet been determined. The removal of the masonry walls between the center tunnel and side tunnels (the plans for which were shown in our issue of Oct. 17) has been in progress for some time, and to facilitate the work the traffic in the side tunnels has been turned onto the tracks in the central tunnel. Whether this crowding of traffic caused any relaxation of the rules regarding spacing of trains is one of the questions which will doubtless be sifted in the official investigation.

THE ACCIDENT ON THE LIVERPOOL OVERHEAD railway, which was noted in our last issue from the daily newspaper dispatches, is more fully described in the English papers which have since arrived. According to the London "Times," as quoted in the London "Electrician," the train to which the accident occurred was about to draw up to the Dingle terminus of the road, which is an underground station approached by a long tunnel, when the fusing of one of the motors set fire to the cars. The cause and progress of the fire is described as follows by Mr. S. B. Cottrell, the engineer and manager of the road, in a statement made to the London "Standard":

> The train to which the accident occurred was the 5 o'clock express from Seaforth, due at Dingle at 5.32. The cause of the trouble was the fusing of one of the motors, which set fire to the rest of the train, and, at a later period, to some sleepers which had been stored in the tunnel for renewal purposes, and probably would have been used in the next fortnight or month, as circumstances required. The fire would have been of a trivial character but for the gale, which was blowing from the westward straight up the tunnel, and practically converted the tunnel into a longitudinal chimney. The gale, in fact, was so strong that at the mouth of the tunnel it was difficult to keep a hat on. But for that, the total damage would not have exceeded £20, and there would have been no loss of life. Immediately the fire was discovered, the station staff acted with commendable promptitude. They telephoned to the generating station to switch the current off the line, which was the right thing to do, in order that the electricity might not cause any further fire. Unfortunately, at that time the flames had caught such a hold on the body of the train, accentuated by the gale, that they got beyond the control of the company's fire apparatus. We had two hydrants on the platform, which were brought into use, and also hand grenades. The first thing that went was the lighting cable, and a joint in the gas pipe also gave way, with the result that the place was plunged into darkness. The smoke from the burning material was intense and suffocating, and the fumes from the rubber round the cables and of the electrical details made it worse. In the darkness, undoubtedly, the unfortunate people lost their way into the station, and this probably led to the loss of life; but as regards those of the passengers who were sacrificed we cannot help believing that there was no reason why they should have been lost, as it would appear to have been through curiosity that they stayed down in the station. The officers who lost their lives sacrificed themselves to duty. Had they made a rush at the first they could no doubt have escaped. As it was, they remained fighting the fire until they were overcome.

A DOUBLE WRECK occurred on the Pennsylvania R. R. near Johnstown, Pa., on Jan. 3. Four persons were killed and several injured. The wreck appears to have been caused by the mistake of an operator in the block-signal tower, who permitted an east-bound passenger train to run into a freight standing on a siding; a few moments later a west-bound freight ran into the wreckage, a sharp curve shutting off the point of the accident from the view of the approaching train.— An east-bound through passenger train on the Baltimore & Ohio Ry. jumped the track at a curve on a steep grade near Glencoe, Pa., on Jan. 1. Two men were killed and a considerable number were injured. The cause of the derailment is not known.

FOUR DAMS IN THE SOUTH were more or less badly injured by river floods on Dec. 29. The damages wrought near Anderson, S. C., and at Columbus, Ga., are described elsewhere in this issue. The failure of a cofferdam at the dam under construction at Tallassee, Ala., described in our issue of Dec. 5, was followed by the destruction of some 125 ft. of closure masonry. The cofferdam was about 50 ft. high and the water, it is said, reached an elevation of 10 ft. on the crest of the completed portion of the dam. The concrete facing had not been applied to the lower side of the portion of the dam which was carried away. A second dam on the Tallassee River, below the one just mentioned, was also badly injured. It was a new rubble masonry structure.

THE SINKING OF THE STEAMER "WALLA WALLA" by collision at sea on Jan. 2 resulted in the loss of about 40 lives, according to the latest reports. The wrecked vessel was owned by the Pacific Coast Steamship Co., and the accident occurred off Cape Mendocino, California. The "Walla Walla" left San Francisco on Jan. 1 for Puget Sound ports, and on the morning of Jan. 2, while in a fog, was run into by an unknown iron bark. A large hole was torn in the steamer's bow, and she sank in half an hour. The number of lost is not yet definitely known, but will probably exceed forty, as stated above.

A 500,000-GALLON ELEVATED WATER TANK is to be erected at Fairhaven, Mass., to take the place of the one that failed last November. According to local papers the tank will be 40 ft. in diameter and 50 ft. high, have a flat bottom and be supported on a metal tower 85 ft. high. It is said that the contract for the tank has been awarded to Riter & Conley, of Pittsburg, Pa.

ENLARGEMENT OF THE NEW YORK CANALS is recommended by Gov. Odell, in his message to the legislature, upon the following lines: (1) The enlargement of the present Erie Canal locks to accommodate barges of 1,000 tons capacity, together with the substitution of single locks of high lift at Cohoes and Lockport, all at an estimated cost of $13,694,540; (2) building of a new 9-ft. channel from the Hudson River to Rexford Flats, and the deepening of the rest of the present canal to 9 ft. at an estimated cost of $15,076,936. Under the New York constitution the question whether the state shall issue bonds to do this work will have to be submitted to popular vote, provided it receives the approval of the legislature.

THE CARTER, GREENE AND GAYNOR habeas corpus appeals were denied by the U. S. Supreme Court on Jan. 6. The Carter case was for the release from the Leavenworth penitentiary of ex-Capt. O. M. Carter, who is confined there by sentence of court-martial on account of the Savannah Harbor frauds. The Greene and Gaynor cases were from relief from the court order requiring their removal to Georgia for trial upon the indictments for conspiracy found against them in December, 1899.

ENGINEERING AND INDUSTRIAL ENTERPRISES AT SAULT STE. MARIE.

One of the greatest centers of engineering activity at the present time is the district known as the "Soo," including the two towns of Sault Ste. Marie, on the American and Canadian sides of the St. Mary's River. This river, forming the outlet from Lake Superior to Lake Huron, has a series of rapids which make it unavailable for navigation, and a lock and canal were built as long ago as 1797 to facilitate the passage of the traders' canoes. This was built on the Canadian side of the river by the Hudson Bay Co. The development of lake navigation rendered the construction of large locks imperative, and at the present time there are three such locks in operation; two on the American side and one on the Canadian side. All these have been described in our columns.

The river has a fall of about 19.3 ft. at the rapids, in a distance of 3,000 ft., developing about 175,000 HP., and some years ago projects were formulated for utilizing a portion of this power. At the present time, one power canal and plant is in operation on the Canadian side, and two other canals and power plants (one on each side) are now in progress of construction. The three plants when completed will develop about 100,000 HP. With cheap power and good navigation and railway facilities, the "Soo" has become a center for numerous and varied industrial enterprises, and in the present article it is proposed to describe in a general way these several enterprises. The general location of these enterprises is shown in the sketch map, Fig. 1, while Fig. 2 is a plan of the industrial works at Sault Ste. Marie, Ont.

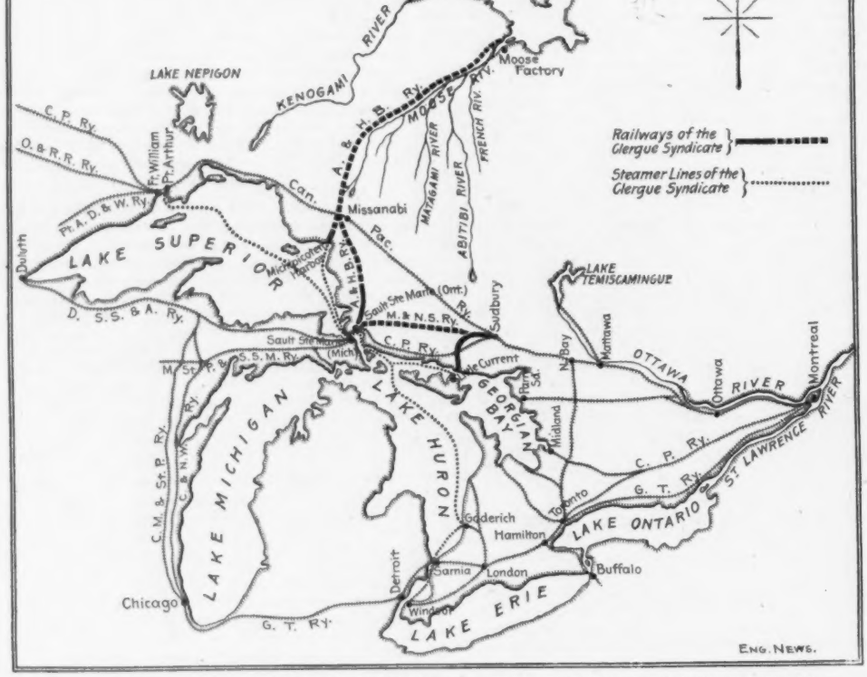


FIG. 1. MAP SHOWING THE ENTERPRISES OF THE CONSOLIDATED LAKE SUPERIOR POWER CO.

In 1894, an American syndicate looking for new investments, began an investigation along the basin of the St. Lawrence River, extending from the Gulf of St. Lawrence to Lake Superior, to ascertain what opportunities offered for water-power development. At Sault Ste. Marie, Ont., it was found that a power canal had been commenced by the city, but that great difficulty was being experienced in getting money to complete it and in getting any company to utilize the power. The syndicate purchased the canal, organized the Lake Superior Power Co., and installed a power plant, but as there was still no outside demand for the power, the company decided that it would undertake the utilization of the power for manufacturing industries of its own. The manufacture of wood pulp for paper-making was first undertaken, using the spruce timber from the forests on the Lake Superior watershed. The first mill had a daily capacity of 20 tons of pulp, but this has been enlarged, and the present capacity is 150 tons of ground pulp per day.

The pulp made in the first mill was "mechanical" pulp, made by abrading the spruce logs with heavy grindstones. This breaks and destroys the fiber, and for paper-making it is necessary to mix a certain proportion of long-fiber pulp to make the paper strong enough to pass over the rolls of a printing press without breaking. This kind of pulp is made by a chemical process, disintegrating the wood by the action of acids without breaking the fiber. The next step taken, therefore, was to establish a chemical or sulphite pulp mill. For this process sulphur is required, and most sulphite mills use sulphur imported from Sicily, which would cost about $25 per ton at the mill in Canada, forming a large item in the cost of manufacture. Investigations were undertaken under the direction of Mr. F. H. Clergue, manager for the syndicate, with a view of obtaining sulphur from a more available and economical source, which was found in the nickel deposits at Sudbury, Ont., as noted more fully below. The reduction of the nickel ore produces sulphuric acid gas and an alloy of nickel which is used in making nickel steel. For the process of separating copper from the nickel sodium is required, and this is manufactured here, the products being a caustic soda and a bleaching powder. This process is also described below.

To supply the necessary amount of wood for the pulp mills a railway was required, and concessions have been obtained from the government for a line from Sault Ste. Marie north to the Canadian Pacific Ry., and on to Moose Factory, on Hudson's Bay. For this the company will receive a cash subsidy and land grants, and is required to colonize the district thus opened up with a certain minimum number of settlers annually. These settlers will occupy lands cleared of timber by the company.

Another step taken by Mr. Clergue was the exploration and development of iron ore resources in the Algoma district (in which Sault Ste. Marie is situated), and great deposits were found about 12 to 20 miles north of Michipicoten, from which point a railway was built to the mines. As trou was experienced in obtaining ships to carry th ore from the docks at Michipicoten to the fu naces at Midland and Hamilton (the furnaces Sault Ste. Marie not being ready), four steame of 2,500 tons capacity were built in England f this service, being of a size suitable for passi through the St. Lawrence canals and locks. Iro and steel works, with various auxiliary plant are now being established, and will soon be read for operation. These works are described mor fully below as the result of a special visit of in spection made by a member of the editorial staf of Engineering News.

During the past summer about 8,000 men were employed, and the weekly pay roll amounted to some $60,000. The expenditures for construction up to the present time have been about $10,000,000, while a similar amount will be required for completing the present works and building the extensions, additions and new plants already provided for.

The entire enterprise is controlled by the Consolidated Lake Superior Power Co., which is backed largely by Philadelphia capital, and has a capital stock of $17,000,000. The water power plants, the railway, the pulp mills, the steel works and other separate enterprises are operated as independent concerns or companies, for convenience in management and bookkeeping, but these companies are all under the direct control of the main company. Mr. F. H. Clergue is General Manager, Mr. E. V. Clergue, Mines Manager, and Mr. B. J. Clergue, Assistant to the President. Mr. A. S. Crane, M. Am. Soc. C. E., is Chief Engineer of the Lake Superior Power Co. (operating on the Canadian side); Mr. H. Von Schon, Chief Engineer of the Michigan Lake Superior Power Co.; Mr. W. Z. Earle, Chief Engineer of the Algoma Central & Hudson Bay Ry.; Mr. John A. Wilde, Chief Engineer of the Algoma Commercial Co.; and Mr. E. A. Sjostedt is Chief Metallurgist.

The enterprise has been conducted on a large scale and with great liberality in regard to explorations, investigations and experiments for the purpose of discovering new resources of raw material and new processes for the manufacture of various products. This work has required the service of large numbers of civil and mechanical engineers, metallurgists, chemists, electricians and specialists, and the equipment of extensive laboratories.

The company owns large tracts of land at Sault Ste. Marie, iron mines at Michipicoten, nickel mines at Sudbury, and grants of timber and mineral lands along its railway lines and their extensions. It has steamer lines from Sault Ste. Marie on Lake Huron to Windsor (opposite Detroit, Mich.), and on Lake Superior to Port Arthur, Ont., with a separate line to Michipicoten. Other steamers are operated for cargo and ore service. It also owns and operates hotels at Sault Ste. Marie and Michipicoten, Ont. Electric street railways are to be built in both cities of Sault Ste. Marie. For power and manufacturing purposes, the company has canals, power plants and industrial establishments on both sides of the river, but principally on the Canadian side.

The combined industrial enterprises at Sault Ste. Marie, on the Canadian side alone, will include the following: (1) a mill for "mechanical" paper pulp; (2) a caustic lime plant operated by the Canadian Electro-Chemical Co.; (3) the Algoma Iron Works; all these are already in operation; (4) a "chemical" or sulphite pulp mill; (5) a nickel ore reduction plant, producing sulphur for the pulp mill and nickel for nickel steel; (6) a complete steel rail plant with blast furnaces, Bessemer converters, and a rolling mill; (7) a by-product coke oven plant; (8) a by-product charcoal plant; (9) an iron and steel pipe and tube plant; (10) an industrial town for the employees at the various works.

The pulp mills, rail mill, machine shop, office building, and several other buildings are built of the local sandstone, much of which is obtained from the canal and other excavations. These buildings have mainly steel trusses supporting slate roofs. The power house at the rail mill is of monolithic concrete construction, and other buildings are of steel frame construction with plank and felt sheathing covered with corrugated iron. The main plants are being designed and built under special contracts, but much of the work on

he Canadian side is being done directly by day bor, under the direction of Mr. Crane. This in-ludes the foundations and general preliminary ork, the ore dock, industrial tracks, etc. Con-ete has been used very extensively for the foun-ation work, and has been laid during severe win-er weather. The materials are heated, but no ime is used, and the results are perfectly satis-actory.

Fig. 3 is a general view of the Canadian works. he large building at the left is the office build-g, beyond which are the "mechanical" pulp mills nd chemical works, the tail race being between his and the office building. Next to the right is he machine shop of the Algoma Iron Works, hile the tall building further to the right is the chemical" or sulphite pulp mill. In front of it s the old block house, and behind it is an inclined overed gallery in which is the conveyor for car-ying the chips from the wood mill to the bins of he pulp mill. The dark tower again to the right is the sulphuric acid plant. The new power canal will cut across the ground in front of this view.

Near the office building is one of the old block houses of the Hudson Bay Co., which has been repaired and fitted up as a residence for Mr. F. H. Clergue, and now stands within private grounds. Close to this is the old lock of the canal built by the Hudson Bay Co., in 1797, for the passage of canoes. The lock is 38 × 8¾ ft., with a lift of 10 ft. It has been cleaned out and repaired, and is now surrounded by a grass plot, but the old boat canal has long ago disappeared.

of the Michigan Lake Superior Power Co. were described very fully by Mr. H. Von Schon, the Chief Engineer, in an article published in our issue of Aug. 4, 1898. Work was commenced in 1898, and the canal is expected to be completed this winter, though the power plant will not be ready until some months later.

The water power rights on the American side of the river were purchased by the Clergue syndicate in 1896. It was originally intended to make a long straight canal, but this was changed to a curved canal much shorter in length, with a power house parallel with the main shore of the St. Mary's River, as shown in the plan published with the article above referred to. The canal will be two miles in length, 164 ft. wide on the bottom, with a depth of 30 ft. of water. At the lower end it will open into a large forebay or stilling basin, where the velocity will be reduced from the 6 or 7 ft. per minute of the canal to 2 ft. per minute. The canal passes through various materials. In solid rock it has vertical walls, the excavation being done by channeling machines, as on the Chicago Drainage Canal. In earth and in the mixture of earth and stone it is lined with plank on 12 × 12-in. mud sills, the latter being laid on piling, and well tamped. The planking is 4 ins. wide, 3 ins. thick on the slopes and 2 ins. on the bottom. Above the water line there is stone paving to the slopes. Fig. 4 is a view of the construction work on the plank lined section of the canal. At the head of the canal will be controlling works, consisting of concrete piers and Stoney gates. Above these works the sides of the canal will flare outward. The canal will be crossed by no less than 13 steel bridges, which are being erected and paid for by the city, the power company building the concrete abutments.

Along the side of the forebay, and opposite the mouth of the canal is the power house, 1,300 ft. in length and 100 ft. in width. This has a monolithic concrete foundation, above which, to the level of the floor of the penstock chambers, it is built of concrete blocks, forming a horizontal arch at each chamber to resist the thrust due to the weight of water in the forebay. The building will be divided into 81 chambers, of which 80 will be for turbines, and one (near the middle) for flood gates or sluices. The partitions between the chambers are of vertical I-beams with concrete filled in between them. The river front of the building is of red sandstone, and the roof is of slate, carried by steel roof trusses.

A peculiar feature of the plan is that the floor of the generating room will be level with that of the penstocks, 16 ft. below the level of the water in the canal and 3 ft. above that of the water in the river. The faces of the penstocks, forming the inner wall of the generating room, will be semi-circular steel shells sprung between the partitions. The upper floor will be for manufacturing purposes, and the Union Carbide Co. has already contracted to take half of the 40,000 HP. generated for a new carbide plant at this point. Alkali works will also be operated. Power will also be transmitted by cables to the Canadian side of the river.

Each penstock will contain two pairs of inward-flow horizontal turbines of 250 HP. mounted tandem on a shaft which extends through the steel shell or wall into the generating room, where the generators will be connected directly to the turbine shafts.

Mr. H. Von Schon is Chief Engineer; Mr. J. H. Brace, Assistant Engineer; and Mr. H. H. Marden, Superintendent of Construction. The contractors are as follows: Canal, E. D. Smith & Co., of Philadelphia, Pa.; forebay and power house, Mason, Hoge & Co., of Frankfort, Ky.; dredging channel above headworks, Hubbell & Co., of Saginaw, Mich.; turbines, Webster, Camp & Lane Co., Akron, O.

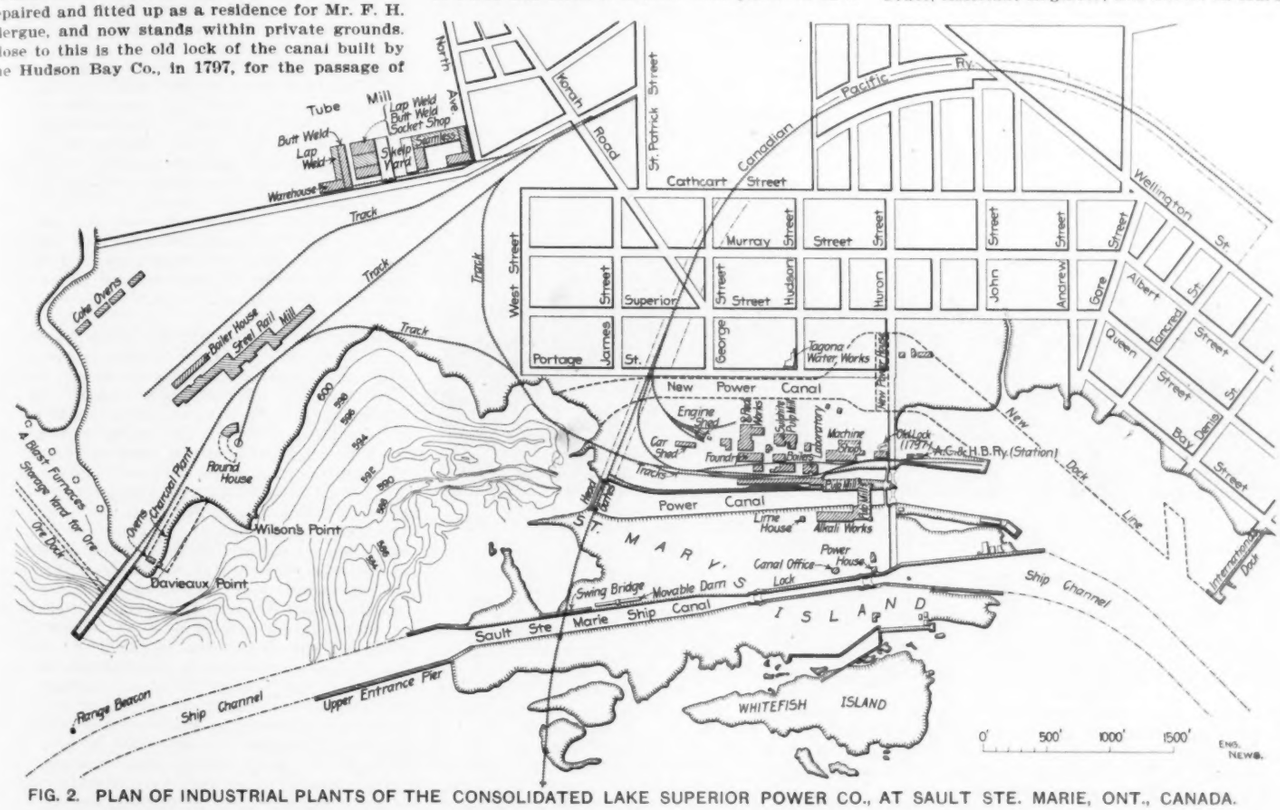


FIG. 2. PLAN OF INDUSTRIAL PLANTS OF THE CONSOLIDATED LAKE SUPERIOR POWER CO., AT SAULT STE. MARIE, ONT., CANADA.

POWER CANALS AND PLANTS.

CANADIAN SIDE.—The first canal was commenced by the city of Sault Ste. Marie, Ont., in 1891, but proved a "white elephant," and was sold to the Lake Superior Power Co. in 1894. The plans were originally prepared by Mr. Zephaniah Malhiot, who was resident engineer; he was succeeded by Mr. Joseph Cozens, and then by Mr. J. A. Wilde, both of Sault Ste. Marie, Ont. This canal is 2,000 ft. long, 125 ft. wide, and carries 14 ft. of water. It has a fall of 19 ft., and can develop 20,000 HP., of which about 16,000 HP. are now utilized by the pulp mills, chemical works, iron works, and the local water supply and electric lighting plant (Tagona Water & Light Co.).

The new canal, whose location is shown in Fig. 2, will be 2,600 ft. long and 150 ft. wide, with 30 ft. of water. It will give a fall of 19 ft., developing a total of 50,000 HP. This plant will furnish power largely for electrolytic reduction works.

AMERICAN SIDE.—The canal and power plant

DAM IN THE ST. MARY'S RIVER.—As the power canals will take a large amount of water from the river, the United States government has required that the company shall build a dam partly across the river channel at the head of the rapids, so as to give a greater flow available for the ship canal and locks. It was at first proposed to build a submerged rock-fill dam, but the government engineers have required that it shall consist of concrete piers with steel gates or shutters, on the Stoney system, so that the flow can be regulated to suit the requirements. This dam will be about 2,000 ft. long, extending from the Canadian side.

PULP MILLS.

The first manufacturing industry established was the pulp mill built by Mr. F. H. Clergue in 1895. This is a stone building, 80 × 300 ft., taking

power to the extent of 7,500 HP. from 24 turbines of the American type, made by the Globe Iron Works, of Dayton, O. These turbines are 51 ins. diameter and are rated at 300 HP. each. Mill No. 2 was completed in 1897. This is also a stone building, 100 × 300 ft., and at present contains 18 McCormick 51-in. water wheels, built by S. Morgan Smith, of York, Pa. This mill supplies the light and water to the city, as well as electric power to the steel plant, iron works, reduction works, sulphite mill, etc.

At first only the wet pulp was made, but as dry pulp could be sold to better advantage a special drying process was devised, in which the pulp sheet is passed over a horizontal drum heated by steam. The finished product is now delivered in rolls of material resembling a coarse blotting paper. The mill contains 22 grindstones and 22 pulp drying cylinders or drums. The mill produces about 100 tons of pulp per day, requiring 75 HP. per ton of pulp. The machinery equipment was built by the Algoma Iron Works (described below) and the output capacity of the mill is about 30,000 tons per year, valued at about $800,000. Much of the product is sent to Japan. The cost of manufacture is exceedingly low, and is less than at most other large plants.

As sulphite, or chemical, pulp commands a higher price than the pulp produced by the mechanical process, it was decided to put in a sulphite mill. This new mill has eight cylindrical wooden vats 5 ft. diameter and 100 ft. high. These are filled with limestone; the sulphurous acid gas

FIG. 3. VIEW OF BUILDINGS OF THE CONSOLIDATED LAKE SUPERIOR POWER CO., AT SAULT STE. MARIE, CANADA.

from the nickel ore roasters passes up through these vats and is absorbed by a descending stream of water, forming a solution which attacks the lime or calcium carbonate and forms a calcium sulphite.

For this process the wood is sliced into chips by knives attached to the faces of revolving steel disks, and these chips are carried by conveyors to bins in the pulp mill. The bins have hopper bottoms fitted with gates, and under each hopper opening is the mouth of a huge steel digester 17 ft. diameter and 54 ft. high, shaped like a Bessemer converter, and holding about 30 tons of chips at a charge. These digesters are lined with acid-proof brick and after the chips have been cooked with steam they are treated with the sulphite solution. Steam for cooking the chips and drying the pulp is furnished by five batteries of Stirling water-tube boilers of 250 HP. each. The machinery for drying and finishing the pulp is similar to that in the "mechanical" mill, and is operated by electricity generated at the water power plant.

Most pulp mills ship the "mechanical" and "chemical" pulp separately, and paper manufacturers have to make their own mixtures. At these works, however, the two styles of pulp will be mixed in the various proportions required for different grades of paper, the product being then ready for the use of the paper makers. This work will be done in the "mechanical" mill, the necessary alterations for which are now being made. Each of the two mills has a capacity for treating about 150 cords of wood per day of 24 hours, or 300 cords in all. One cord of wood produces nearly one ton of pulp.

NICKEL REDUCTION WORKS.

It has already been shown that most of the sulphur for the sulphuric acid used in the sulphite pulp mills in this country is imported at considerable cost. As the nickel deposits of Sudbury, Ont., comprise a combination of nickel, sulphur, copper and iron, Mr. Clergue conceived the idea of obtaining the sulphur from this source. Special investigations were made and it was found that in the reduction of the nickel ore by processes then in use great quantities of sulphurous gas were being wasted by being discharged into the open air. Up to that time no practical method had been discovered for recovering sulphur from this pyrrhotite ore, but special research was made under Mr. Clergue's direction, with the result that a new and commercially practicable process was developed by which large quantities of sulphuric acid gas may be manufactured and further reduced into liquid sulphuric acid, which may be shipped to other pulp mills.

As a result of this, nickel mines and properties were purchased at Sudbury and a plant erected at Sault Ste. Marie for extracting sulphurous acid gas from the pyrrhotite ore. The residue contained a combination of nickel and iron, which when treated by an electrolytic reduction process produced an alloy of nickel steel of such quality that the Krupp works, of Germany, made a contract for a supply for five years. It was found, however, as the mines were worked, that while a large proportion of the ore was nickeliferrous pyrrhotite, a still larger proportion contained copper pyrites, and it became necessary to devise means for reducing and separating the copper and nickel. For this also a new process was devised. In this latter process sodium is required, and this is manufactured in the alkali plant described below.

In the electrolytic smelting process devised for separating and reducing the nickel, the ore is ground to powder and passed downward through a roasting furnace of special design, from which it is taken out at the bottom as an oxide of iron and nickel, with copper in combination. The gas given off during the roasting is piped to the calcium sulphite plant already noted. The reduction works include the reduction of the ore to produce both sulphur and ferro-nickel, and the separation of copper from the latter product, the refined ferro-nickel containing about 6% of nickel. It is this refined product which is supplied to the Krupp steel works of Germany, as noted above.

ALKALI PLANT.

One-half of the building which contains the mechanical pulp mill is occupied by the alkali works, which are operated by the Canadian Electro-Chemical Co. By the Rhodin electrolytic process, using a mercury cathode, 9 tons of bleaching powder and 5 tons of caustic soda are made daily by the apparatus installed. These works are now in regular operation.

Here the sodium required for the process of separating the copper from the nickel is manufactured, using salt from Windsor or Goderich, Ont., as the raw material, from which caustic soda and chlorine gas are produced by Rhodin's electrolytic process. The gas is a by-product, and to utilize it a bleaching powder plant was put in. The powder consists of about 44% of active chlorine gas and 56% lime, the lime serving as a medium to absorb and retain the gas. Some of the gas, however, is forced into lime water under pressure, and this liquid is then used for bleaching the sulphite pulp.

STEEL WORKS.

The steel works are of considerable size, and will include four blast furnaces, two Bessemer converters and a rail-rolling mill. An open-hearth steel plant will also be built later on. Two of the blast furnaces will use coke fuel, and will be 2[illegible] ft. diameter and 90 ft. high. The other two will be for charcoal, and will be 14 ft. diameter and 7[illegible] ft. high. The furnaces will be equipped with mechanical charging appliances, and each will have a capacity of producing 150 tons of iron per day. There will be 30 Wellman-Seaver hot blast stoves. The Bessemer converters have acid linings, and are of 5 tons capacity. Work is in progress on the concrete foundations for the furnaces, but as the rail mill will be ready for operation some time before the furnaces, three cupolas have been built to supply molten pig metal for the converters temporarily. These cupolas are 8 ft. diameter, and each has a capacity for melting 50 tons of iron per day of ten hours. The pig iron will be principally supplied by the Canada Furnace Co., of Midland, Ont., which uses the company's ores. For the blast for the furnaces, cupolas and converters there are two 600-HP. blowing engines, one built by the Allis-Chalmers Co., of Chicago, and the other by the Southwark Foundry & Machine Co., of Philadelphia, Pa.

The rail mill is a handsome building with walls of the local red and white Potsdam sandstone, and steel roof trusses, carrying a plank sheathing and slate covering. It is 1,500 ft. long and 80 ft. wide, with two offsets. Fig. 5 is a view of the building taken some months ago, showing the style of masonry construction with steel roof trusses. The two separate walls at the left are those of the boiler house. Concrete has been used very extensively for the foundations of the mill, blast furnaces, heating pits and machinery, and for this purpose a special concrete mixing plant, with mixers of the cubical box type, was erected on the site.

In this one building all the operations will be performed. There will be a two-high roughing mill, and a finishing mill, each driven by a Porter-Allen engine of 1,500 HP., with cylinders 40 × 48 ins. From the converters, the metal will be poured into ladles and thence into molds mounted on trucks or buggies running on a narrow gage track. The ingots will be stripped by an overhead stripping crane, which also has an auxiliary trolley hoist for charging the ladles on the ladle crane. They will be charged into two four-hole soaking pits, from which they are delivered as required to the blooming tables by an electric traveling crane, the same crane carrying the blooms to the rolls. The ingots will be passed through a 32-in. blooming mill, sheared to length and charged into four Siemens, regenerative heating furnaces. Another electric traveling crane is provided for handling the rolls when sections are to be changed. The roll trains and transfer tables will be driven by electricity. From the saws, cooling beds and cambering machine the rails pass to the loading beds, being handled by pneumatic hoists. The crane runways are supported on steel columns of plate girder construction, independent of the building. The boiler house is also a stone building, and contains a row of 16 Stirling boilers aggregating 4,000 HP. Electricity for the cranes, transfer tables, etc., will be furnished from a power plant contained in a separate building, which is of monolithic construction, with walls 25 ft. high. Current will be transmitted at 550 volts from the main generating power house on the canal, and transformed down to 220 volts in the concrete substation at the mill.

The mill will make rails 30 or 60 ft. long, and of weights up to 100 lbs. per yd., the Am. Soc. C. E. sections being adopted. Structural shapes will also be made later. The capacity of the mill is 600 tons of rails per day. The company has a contract with the Canadian government for supplying 35,000 tons of rails per year for five years, and delivery was to have been commenced in August, 1901. The steel strike in the United States and other difficulties, however, delayed the work, and the government therefore purchased rails in England, charging them to the company. It is expected, however, that the mill will be turning out rails very soon.

The design and construction of the entire plant of the steel works has been in the hands of the

Wellman-Seaver Engineering Co., of Cleveland, O. The contract for the masonry and concrete work was let to H. E. Talbot & Co., of Dayton, O.; and the steel construction to the Riter & Conley Co., of Pittsburg, Pa. Mr. D. D. Lewis, formerly of the Lorain Steel Co., is General Superintendent of the steel works; Mr. F. N. Waterman, formerly of the Dominion Steel Co., is Mechanical Engineer.

ORE DOCKS.

For receiving and storing the iron ore brought down by lake steamers, an ore dock 2,150 ft. long and 380 ft. wide is now being built, near the blast furnaces. This is of pile and timber construction. Upon it will be an ore unloading and conveying plant for taking the ore from the steamers or barges and forming the stock piles under the conveyors, which will have a span of 200 ft. between the supporting legs. The dock will carry about 1,000,000 tons of ore in the stock piles. Along the frontage of this dock, six large dredges have been at work dredging out a channel 21 ft. deep to accommodate the largest lake vessels. These dredges work in a row, each one dredging against the breast left by the one ahead. The dock is being built by the Lake Superior Power Co., but the dredging has been let to a number of contractors.

At Michipicoten Harbor is an ore-loading dock with 12 pockets, the length of the dock and trestle being about 500 ft. The ore is brought down from the mines in the steel cars of 100,000 lbs. capacity and discharged through the pockets and chutes into the vessels. The pockets are not used for storage at present, but the ore is held in trains until the vessel is ready to receive it, this method being considered as the more economical in view of the short run between the mine and the harbor. For this purpose, a storage yard with tracks accommodating 12 cars each has been built near the dock, and a train of 12 cars can be quickly run onto the dock and discharged into the 12 chutes.

BY-PRODUCT COKE OVENS.

The coke ovens will be built by the United Coke & Gas Co., of New York, and will be of the Otto-Hoffman by-product type. This plant will comprise about 168 ovens.

BY-PRODUCT CHARCOAL OVENS.

In two of the blast furnaces charcoal will be used for fuel, as already noted, and each furnace will require about 600 tons of fuel per day. The charcoal for this purpose will be produced in by-product ovens under a new system patented by Mr. Oscar Daube, of the North American Carbon & Chemical Co., of 309 Broadway, New York.

The first battery of four ovens with the necessary condensers, etc., for recovering the by-products, has been completed and will be used for experimental work, but the plans provide for a plant consisting of 28 batteries of four ovens each, with a total capacity of 600 cords of wood per day. The present plant has a capacity of 200 cords of wood every eight days, two days being required for carbonization and six days for cooling. This charge of wood will yield 10,000 bushels of charcoal. The by-products recovered per cord of wood will include 7 gallons of wood alcohol (95% pure), and 160 lbs. of acetate of lime, besides the tar and the gases utilized for fuel. The ovens are large rectangular brick chambers, 36 × 70 × 14 ft., with flues in the sides for carrying off the gases from carbonization, and flues under the floor for carrying (under pressure) superheated non-combustible gas brought directly from the producer into the oven, where it comes in contact with the wood. The temperature in the oven is not allowed to exceed 650° F., so as not to allow decomposition of the gases. The permanent gases, recovered in the condenser, are fed to the superheating furnace as fuel, and these gases form 75% of the fuel required, coke being used only at the beginning of the process and to maintain a uniform temperature. The carbonization takes place in 48 hours, and the oven is left to cool off for six days. With the complete plant, the batteries will of course be operated in series.

Under each oven runs a tunnel, the crown of which rises through the floor and has openings on either side, through which the charcoal will be dropped into the tunnel, in which will be a link-belt conveyor carrying the coke to the storage bins. From the bins the coke will be loaded automatically as required into steel cars running on a narrow gage track to the blast furnace, where it will be charged by machinery, so that there will be no handling of the coke between the oven and the furnace. Fig. 6 shows the ovens, at the right, with the producer in front and the condensers in the background. In this view, Mr. Daube is standing by the producer furnace.

A small plant, with one battery of four ovens, having a capacity of 35 cords of wood per day, is now in operation in New York. The new experimental plant is being built in connection with the Lake Superior Power Co. The size and capacity of the charcoal blast furnaces will necessitate an immense supply of charcoal, which, it is claimed, could not be produced from the ordinary beehive ovens or by the retort process. A number of American furnaces are said to be negotiating for this new process, which will produce large quantities quickly and will at the same time recover the by-products.

ALGOMA IRON WORKS.

This plant comprises a foundry, machine shop and blacksmith shop. The machine shop is of steel frame construction with stone walls, and is 80 × 160 ft., but is soon to be lengthened to 320 ft., to make room for new equipment to enable the shop to deal with the amount of work required for the various plants now in progress of construction or extension. One of the latest tools installed is a large bulldozer for making the deep reinforced angle bars for the Bonzano rail joint, which is the standard adopted for the railway company's lines. The heavy machinery is installed on the floor, while on a gallery are the lighter tools and the tool room. The main line shafting is driven from a 100-HP. electric motor.

PIPE AND TUBE MILL.

This plant will be for the manufacture of wrought iron and steel pipe and tubing, and is being designed by S. V. Huber & Co., consulting engineers, of Pittsburg, Pa. The building will be of steel and stone construction, and the processes of manufacture will include some new features and machinery.

INDUSTRIAL TOWN.

This is located north of the sites reserved for manufacturing enterprises, and is being built for the use of the employees at the various works. Already 130 cottages have been built, and a boarding-house with accommodation for 200 persons. The water supply is furnished by pumping from the river, and a sewerage system is being put in. The sewers of this town and of the various parts of the works will drain into a main circular outfall-sewer of 6 ft. diameter. This is of concrete, with a single course of brick lining for the invert. The contractors for the sewer work are Battle, Braas & Co., of Thorold, Ont.

ALGOMA CENTRAL & HUDSON'S BAY RY.

The main line of this railway now extends from Sault Ste. Marie, Ont., to Goulais, 31 miles, but the grading is completed for about 27 miles further north. A branch to Michipicoten will give a rail outlet for the iron mines of the company. The railway is well and substantially built, and many trestles are being filled in, extensive gravel pits being worked by several steam shovels, and large gangs of men are at work upon the track and general improvements. A considerable amount of yard work, spur track construction, etc., is required to accommodate the various works in progress at Sault Ste. Marie. The tracks are laid throughout with 85-lb. rails of the Am. Soc. C. E. section, spliced with the Bonzano rail joint, the heavy angle bars for which are made at the shops of the Algoma Iron Works, already noted. All trestles have double-guard timbers, the inner timbers being faced with angle iron. Ballast trains are unloaded by plow and cable, but some of the 50-ton steel hopper-bottom ore cars are being used for distributing gravel. The maximum grades are 1½%, and the sharpest curves are of 12°.

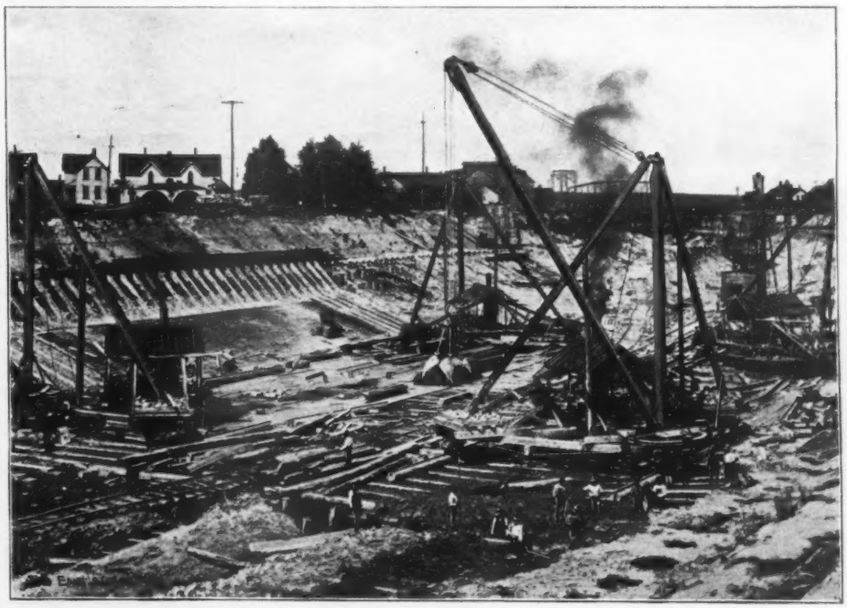


FIG. 4. EXCAVATING POWER CANAL FOR THE MICHIGAN LAKE SUPERIOR POWER CO., AT SAULT STE. MARIE, MICH.
H. Von Schon, Chief Engineer. E. D. Smith & Co., Philadelphia, Pa., Contractors.

The Goulais River bridge will be a steel structure of 180-ft. span, on concrete abutments. The ground is a clean coarse gravel, forming a good foundation. A cofferdam of Wakefield triple-lap sheet piling is built for each abutment, the material within being excavated by hand and loaded into skips handled by a steam derrick. The water is pumped out by a horizontal centrifugal pump at the bottom of the excavation, the vertical shaft being driven by a belt from a portable engine.

At Sault Ste. Marie are a passenger and freight station, piers for shipping, an 8-stall stone roundhouse, two 75-ft. steel plate girder turntables (built by the Hamilton Bridge Co.), a water tank, and other facilities. The equipment includes some very large twelve-wheel and consolidation locomotives, of 140 tons (with tenders), and the Pressed Steel Car Co.'s hopper bottom ore cars of 100,000 lbs. capacity, weighing 29,000 to 29,300 lbs. empty. These cars are all fitted with M. C. B. couplers and air brakes, and have journals 5 × 9 ins. For passenger and general freight traffic there are lighter engines and ordinary freight cars.

This line is now being extended northwest from the Goulais River towards the Canadian Pacific Ry., near Amyot, a distance of 150 miles from SaultSte.Marie. Much of the work here will be very heavy, including rock tunnels and cuts. Eventu-

ally the line will extend north of the C. P. Ry., following probably the valley of the Moose River, the objective point being Moose Factory on Hudson's Bay. Much of this part of the line will be in a flat or rolling country, with few engineering difficulties. Land and timber grants have been given to the company by the Canadian government, and it is expected that considerable mineral wealth may be found. In fact, explorations of this region were made by the engineers of the Ontario government in 1900, and are now being made for this purpose by the company's engineers. Most of the timber will be cut for use in the pulp mills.

FIG. 5. STEEL MILL UNDER CONSTRUCTION; SAULT STE. MARIE, CANADA.

An independent railway, 12 miles long, extends from Michipicoten Harbor north to the iron mines, and this is also of substantial construction. There are very few trestles, the line being carried across the numerous deep ravines by high embankments having concrete waterways. There is one large steel bridge over the Magpie River, having three plate-girder spans on concrete piers. The passenger and general freight traffic are inconsiderable, but the ore traffic is already heavy, and is carried over the line in cars of 100,000 lbs. capacity built by the Pressed Steel Car Co. The grades are almost entirely in the direction of the traffic, and are 1½% maximum, with curves of 12°. The ore trains consist of about 13 cars, hauled by consolidation and twelve-wheel locomotives weighing about 130 tons. The ore dock at the harbor has already been described. This road will eventually be extended to connect with the main line from Sault Ste. Marie.

A third line, 80 miles long, extends from Little Current, north of Manitoulin Island, to Sudbury, and is for the purpose of bringing nickel ore from the company's mines at Sudbury down to the coast, to be loaded into vessels for transportation to the nickel reduction works at Sault Ste. Marie. This road was built as the Manitoulin & North Shore Ry., but was acquired by the Lake Superior Power Co. as an independent outlet for its nickel mines. It is isolated from any connection with the other parts of the company's railway, but will eventually be extended westward to Sault Ste. Marie, so that the ore may be carried from the mines to the reduction works either by rail or water. With the charter for this line the company has a cash subsidy of $1,000,000 and a land grant of 3,000,000 acres.

IRON MINES.

The iron mines are at Michipicoten, about 150 miles by steamer from Sault Ste. Marie, and thence a 12-mile railway extends to the Helen mine. This is a great surface mass of ore about 1,000 × 400 ft., and 200 ft. deep. It is largely hematite, with some pockets of pyritic ore. A hill of almost solid ore is being excavated by hand and by steam shovels, being very soft and covered by only about 12 ins. of moss and earth. At deeper levels the ore is drilled and blasted, the excavation forming a huge open pit or crater. A lake of about 24 acres in area has been lowered 12 ft. by draining to another lake below it, and is now being pumped dry by centrifugal pumps to enable the bottom to be worked in the same way. The ore averages 58.63% iron, 0.091% sulphur, and 0.125% phosphorus. At present the ore is being shipped to Buffalo, N. Y.; Newcastle, Pa.; and to Midland and Hamilton, Ont. About 50,000,000 tons of ore are said to be in sight.

Various methods of excavating and handling the material are in use. In the soft ochreous earth material, the excavation is partly by steam shovel and partly by pick and shovel. The steam shovel loads directly into the 50-ton steel cars, or into four-wheel side-dump cars. With hand work, the men with wheelbarrows load a small car which stands under a platform, and runs out onto a trestle and is dumped into the large dump cars. These dump-cars are hauled by locomotives to a dumping station above the track about half a mile from the mine, where they are dumped into side chutes leading to pockets directly over a loading track for the 50-ton main line cars. In the rock excavation, the material is handled partly by revolving cranes with very long latticed booms, which carry the skips and dump them on a platform having a hole in the center, under which is a small car running on an inclined plane. At the head of the incline the rock is dumped into bins or pockets from which chutes lead to the 50-ton cars standing on a loading track above the pit, but below the pockets. There is also a cableway carrying skips and dumping them into the same pockets at the head of the inclined plane. Two ore crushing plants, with Gates crushers, have been installed, and have a total hourly capacity of 600 tons.

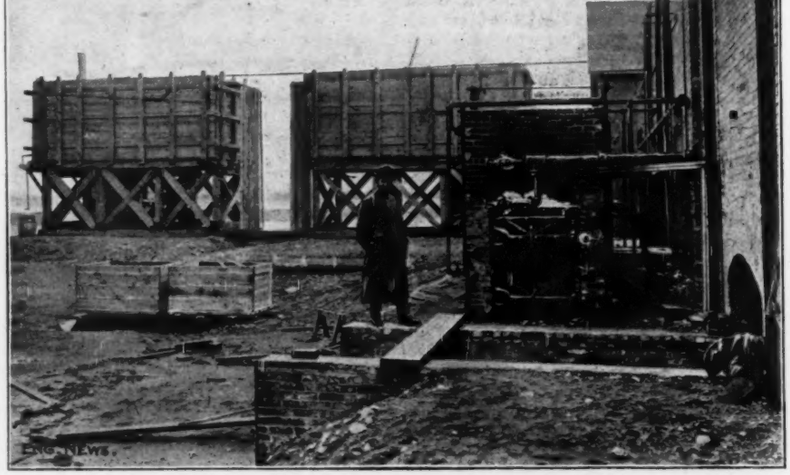


FIG. 6. BY-PRODUCT CHARCOAL OVENS AT SAULT STE. MARIE, CANADA.
Oscar Daube (North American Carbon & Chemical Co.), 309 Broadway, New York, Inventor.

About 10 miles beyond is the Josephine mine, which is newer in development, and consists of underground workings. The ore is a high-grade hematite, suitable for Bessemer work. It averages about 66% iron, 0.025% phosphorus, 5% moisture and 0.02% sulphur. Other mines will be developed as required.

NICKEL MINES.

The company's mine in the pyrrhotite ore at Sudbury, Ont., is now capable of producing about 300 tons of ore per day. Other nickel mines are owned by the company along the line of its Manitoulin & North Shore Ry., and these will be developed as required. The reduction of the ore and the production of sulphur, ferro-nickel and copper, have already been described. The average analysis of the ore is about as follows: Nickel, 4.25% copper, 1.40%; sulphur, 33%.

ANNUAL MEETING OF THE AMERICAN ECONOMIC ASSOCIATION.

The fourteenth annual meeting of the association was held at Washington, D. C., Dec. 27 to 30. The presidential address, by Dr. Richard T. Ely, of Madison, Wis., was on "Industrial Liberty." Prof. Chas. A. Tuttle, of Wabash College, read a paper on "The Position of the Workman in the Light of Economic Progress," and Mr. Herman Justi, Commissioner of the Illinois Coal Operators' Association, discussed "Conciliation and Arbitration Among Miners." The report of the Committee on Uniform Municipal Accounts and Statistics was presented by M. N. Baker, of the editorial staff of this journal. It consisted, virtually, of a resubmission of the report of a year ago, which, coming the very last thing on the program, received no discussion. The supplementary report stated that the conclusions of the original report, so far as they related to the need for uniform accounts and statistics, had been pretty generally accepted, while the one relating to co-operative efforts had been not only well received, but several steps had been taken to put its suggestions into effect. The matter of State control and the publication of statistics might well be left until the "movement has attained more definite shape and greater volume and force than could be expected in so short a time." It is pertinent to inquire what the Census Office might best undertake in the way of municipal statistics. The report concluded as follows:

Since the Detroit meeting of this association, a committee of the National Municipal League, of which two members of your committee are likewise members, has been formed. This committee is actively engaged in an attempt to formulate a general system of uniform municipal accounting. It is also trying to bring into unity the efforts of various special societies, like water-works associations, to perfect either schemes of accounting or summaries of statistics for annual reports. The work of the National Municipal League committee, up to May, 1901, is outlined in a published report, copies of which are available here or may be secured from the members of either committee. The league hopes that by the date of its next annual meeting it may be able to present both a series of schedules for municipal accounting and summaries of the statistics of many branches of the municipal service.

Your own committee requests the members of the association to co-operate in the general movement by aiding in the formulation of schedules and by trying to induce

city officials generally to apply to their respective municipalities and departments the schedules as formulated. Much of the work is necessarily tentative, and its value cannot be judged until the proposed form of schedules has been tested by actual practice.

The discussion on the report was opened by Mr. Chas. H. Curtis, of New Haven, Conn., who thought that if the effort to secure uniform statistics had to overcome friction it would be from franchise companies. Statistics relating to such companies should be impartial. It has become a fad, Mr. Curtis said, to condemn semi-public companies and to commend municipalities. Mr. Roland P. Falconer, of Washington, D. C., called attention to the chaotic condition of records which vitally affect the inhabitants of cities. He favored immediate State supervision of municipal accounts, without waiting for uniformity.

Mr. Harvey S. Chase, of Boston, and Mr. C. R. Woodruff, of Philadelphia, spoke of the recent work at Chicago on a new system of accounts for that city, following as far as practicable the tentative plan submitted last May by the committee of the National Municipal League. Dr. Wm. M. Hartwell, of Boston, stated that municipal statistics, like generosity, should begin at home. The itch for comparative statistics is bad, at least when in a single year, as at Baltimore not long ago, three estimates of population, with a variation of over 100,000, were used in certain tabulations.

SOME COMPARISONS OF STANDARD SPECIFICATIONS FOR TESTING PORTLAND CEMENT.

By E. Duryee.*

The writer has always considered the Russian specifications for Portland cement excellent on account of the rational requirements they fix as to chemical composition and age before testing, and because of the details they give for securing uniformity in the results of tests made by different testers. Some of these points are brought into comparison with the American, German and French rules in the following notes.

CHEMICAL COMPOSITION.—It is well known that any brand of cement varies somewhat in its analysis. While each brand has a standard analysis which the manufacturer aims to approximate, still on account of occasional variations in the raw materials the percentage of lime is sometimes intentionally changed slightly in order to facilitate the burning of perfect clinker. In addition to these changes trifling variations occasionally happen through some imperfection in the milling of the raw materials. The analyses given in the accompanying table have been collated to illustrate this point.

Table Showing Variation in Analysis of Different Samples of Several Brands of Portland Cement.
(Compiled by E. Duryee.)

Brand and country where made.	Lime.	Silica.	Alumina.	Ferric Oxide.	Magnesia.	Sulphuric acid.	Alkalies.	Moisture.	Total.	Analyst.
Alsen (Germany)	59.98	24.90	11.22		0.38	0.86	0.50	2.16	100.00	....
Alsen (Germany)	63.67	21.13	8.37	2.41	1.36	1.77	.....	.....	98.61	....
						1.81	$\{(NaO)0.631$ $(K_2O)0.517$	$(CO_2)1.14\}$ $(H_2O).270\}$	100.068	C. F. McKenna.
Atlas (Pennsylvania)	60.95	21.30	7.65	2.85	2.95					
Atlas (Pennsylvania)	62.12	22.88	10.43		1.73	0.85	.....	.....	98.01	Atlas Company.
Lafarge (France)	58.38	31.10	4.43	2.15	1.09	0.60	(K_2O) .35*	$(CO_2)1.28$	99.97	C. F. McKenna.
Lafarge (France)	58.38	27.38	2.61	1.02	0.46	0.43	.....	.....	.....	Atlas Company.
White Bros (England)	59.00	23.567	10.02	3.18	1.067	1.45	2.10	.....	100.384	E. Duryee.
White Bros. (England)	58.49	25.8	14.65		0.00	1.00	.....	.....	99.94	E. Duryee.
Knight, Bevan & Sturge	61.04	23.30†	11.50		0.86	1.45	.....	$(CO_2-)0.85$§	100.00	K.,B. & S. circlr.
Knight, Bevan & Sturge	61.38	19.75‡	7.48	5.10	1.28	0.97	0.75	2.92	100.19	....
Colton (California)	59.92	22.25	13.15		0.90	1.20	2.00	.....	99.42	E. Duryee.
Colton (California)	61.484	19.7	14.7		0.73	1.50	1.80	.....	99.814	E. Duryee.

*(NaO) 0.59. †Insoluble, 0.88; made in England. ‡Insoluble, 8.65; made in England. §Organic — 0.12.

The Russian specification states that "the proportion of the sum of the calcium oxide and alkalies to the sum of the silica, alumina and ferric oxide, must in Portland cements be not less than 1.7 to 1, nor more than 2.2 to 1, except in Portland cement made from natural marl, the above proportions may, until further experiments show their inadvisability, be raised to 2.4 to 1, providing the cement will fulfil the usual requirements." If this rule be applied to the above analyses, it will be found that with the exception of the two White Bros. and one of the La Farge analyses, they answer the requirements; in all the others, the modulus falls between 1.7 and 2. The specification establishes a standard by which a cement tester can readily judge whether an analysis of a given cement is satisfactory, and in specifications calling for an analysis it would be well to embody this as a part. The recent report of the Board of Engineers of the U. S. Army on the properties and testing of hydraulic cement (Eng. News, Sept. 12, 1901), recommends that the lime be 1.7 times as much as the sum of the elements that give the cement its hydraulic qualities. The French specification, "cement containing less than 44 parts of silica plus alumina to 100 of lime is also considered suspicious," appears to be too widely divergent from standard analyses of Portland cements to be of any practical use.

NORMAL CONSISTENCY AND SETTING TEST.—There is a general lack of uniformity as to the percentage of water used in making mortar for testing purposes in the different laboratories in this country and in different countries. The latest recommendations in this country come from the recent report of the Board of Engineers of the U. S. A. on the properties and testing of hydraulic cement. This report names the per cent. of water in mortar for all Portlands at 20 for the setting test and 20% for the neat briquettes; the initial set to be determined with a 1-12-in. needle loaded to weigh ¼-lb. Twenty per cent. of water for the setting test mortar will not agree with the normal percentage as determined by the German, French or Russian specifications. It is an arbitrarily fixed proportion not in accordance with general practice, and its enforcement would cause the rejection of many good Portlands.

The German, French and Russian specifications covering this point are as follows:

GERMAN.—The requirements of the Prussian Minister of Public Works read:

> The amount of water used should be such as to require several taps of the plate on the slab to cause the cement to spread to the edges; usually 27% to 30% of water will give this result. The temperature of the room and of the water should be from 15° to 18° C. A normal needle of 1 sq.mm. section (1/8-in. diameter) weighing 300 grammes (10½ ozs.). A metal ring 4 cms. (1⅝ ins.) high and 8 cms. (3⅛ ins.) inside diameter is placed on a glass and filled with cement mortar of the above consistency. The time at which the needle ceases to pass entirely through the cement is called the initial set. The time at which the needle ceases to make a noticeable impression represents the time of setting.

FRENCH.—The room and the water used are to be at a temperature of 15° to 18° C.; the water and cement to be mixed 5 mins. with a trowel of stated dimensions, and the proportion of water to be such that a small quantity of the mixture will fall from a trowel without leaving any adhering portions, and if dropped from a height of about 20 ins. will retain its form and show no cracks. The apparatus for determining the setting-time is the same as the German. Cement which begins to set in less than 30 mins., or sets in less than 3 hrs. is rejected.

RUSSIAN.—The specifications for the normal consistency and setting time are as follows:

> Portland cement must be slow setting, that is, it must not begin to set within 30 mins. from the time the water is added, nor must it be completely set in less than 1 hour, nor require more than 8 hours. The water used in these tests must be fresh water and the mortar must have the normal consistency. The normal consistency of Portland cement mortar and the required amount of water is to be determined by a consistency apparatus consisting of a shaft 1 cm. in diameter weighted to 300 grms., and a box 8 cms. diameter and 4 cms. high to contain the mortar. The shaft is to be pivoted with a millimeter scale.
>
> The determination of the normal consistency of Portland cement mortar, is made by taking 400 grms. of the cement and enough water to obtain a somewhat stiff mortar, which must be thoroughly worked over, and then without any shaking, if possible, put into the cylindrical box which must be placed on glass or other non-absorbent surface, and the excess of mortar removed with a straight edge. The shaft of the apparatus is to be carefully lowered until it touches the surface of the mortar. It is then released and allowed to sink into the mortar. By the right percentage of water used for making the mortar the shaft will sink to within 6 mm. of the bottom. The percentage of water varies in different Portland cements between 24% and 31%, but is found to be generally fairly constant in cement from the same factory.
>
> To ascertain the time of setting, initial and complete set, weigh out 400 grms. of cement mix the required percentage of water, place the mortar in the cylindrical box, bring the box and cement mortar with the glass plate upon which it rests, under the Vicat needle (1 mm. section loaded to 300 grms.) and allow it to sink, at intervals of 5 mins. or more, into the mortar, selecting a fresh place for each penetration. The initial set begins when the needle fails to penetrate the mortar and remains between 0 and 1 on the scale, or in other words remains about ½ mm. from the glass plate. The complete set is indicated when the needle fails to penetrate the mortar and the indicator remains at the 40 mm. mark.
>
> A preliminary setting test may be also made by making a pat on a smooth, clean glass plate of mortar of the normal consistency, 8 to 10 cms. in diameter and 1 cm. thick in the middle. This may be considered set when a gentle pressure of the finger nail fails to make any impression and a gentle rubbing with a trowel or spatula fails to bring water to the surface. In each case the setting time must be determined on at least two pats.

The manufacturer will naturally make his tests, if possible, according to the practice followed in the laboratory where his cement is tested for acceptance. In this country he often finds that the reports from the several laboratories where his contracts call for tests to be made, vary largely on the same cement, both as to setting time and tensile strength. In the writer's laboratory the custom is to add 32% by volume of water of the weight of cement, or one fluid ounce of water to three avoirdupois ounces of cement, for the setting test, and 18% of water, by volume, of the weight of cement used in making briquettes. This rule is followed simply because it is the practice of the cement-testing department of the Los Angeles City Engineer's office, where most of the cement-testing for Southern California, outside the writer's laboratory, is done. The normal consistency of the California Portland cement, as determined according to the Russian specification, calls for 25% water by weight for the setting test. This normal percentage will vary with different brands, and will in the same brand vary somewhat in accordance with the age of the cement.

SULPHATE OF LIME AND TESTS FOR FREE LIME.—Recently engineers and cement testers have expressed fears lest some Portland cements that have passed the hot water test for free lime may yet contain an injurious amount of free lime. The general custom among cement manufacturers of adding sulphate of lime in amounts of about 2% to regulate the setting time also disguises the presence of free lime in a measure. Russian specifications allow the presence of sulphuric acid to the amount of 1.75 (= 2.9% sulphate of lime). The French standard of 1891 fixed the limit at 1% sulphuric anhydride (= 1.7% sulphate of lime). The German manufacturers' association allows the addition of 2% sulphate of lime. The U. S. Board of Engineers' report recognizes the general presence of 2% sulphate of lime in commercial Portland cements.

The results of the writer's experience in testing cement, with and without the addition of small percentages of sulphate of lime, has convinced him that the practical results favor the use of plaster for cement to be used under ordinary conditions. Better tests are secured, both as regards setting time and strength in long and short-time periods, when it is used. The addition of plaster to cement to be used in sea water constructions is still a matter under investigation.

The manufacturer can judge whether a cement is sufficiently cured to be safe for use, and it is more to his interest than that of anyone else to have the cement satisfactory. It is presumable that he will use all care to insure its reliability. Russian specifications make sure of this care by stating that,

> In all cases where the amount of cement to be used for any work reaches or exceeds 10,000 bbls., the superintendence by a government officer, of the factory making the cement, shall be obligatory, in addition to the usual tests.

The writer suggests that the thermometer test be made in connection with the hot-water test for free lime. If at the time the pat is made for the boiling test, an equal amount of cement is made into mortar and a thermometer placed in it, the presence of free lime will be indicated by the rise of temperature. If the cement has been perfectly cured there will not be any change of temperature. Sometimes a cement that has had sulphate of lime added to it will stand the boiling test, but will show considerable heating under this test. If the mortar is made only damp by adding 16% water, the mercury will rise much higher than if it is made quite wet by adding 32% water. The rise in temperature occurs within two minutes after the water is added, and the test can, therefore, be made very quickly. The following table shows the results of some thermometer tests for free

*Chemist California Portland Cement Co., Colton, Cal.

loam and sand, with a light unfertile clay on the higher ridges. This soil has a depth in places as great as 40 ft., and is generally underlaid by a stratum of clay of several feet in thickness, resting upon a stratum of sand, which is itself underlaid by a stratum of hard blue clay belonging to a geological formation long antecedent to the present. The soil is covered by many varieties of large and valuable timber, and is subject to the highest state of cultivation.

The levee of the Lower St. Francis Basin, beginning at the foot of Little Prairie Ridge, near New Madrid, and extending southwardly along the west bank of the Mississippi River to the mouth of the St. Francis River, will prevent the escape of flood water from the Mississippi River into the Lower St. Francis Basin, and reclaim that vast alluvial territory for agriculture, commerce and industry.

Levees have existed in the Lower St. Francis Basin since the first habitation of that territory by man, for he found it as necessary to protect his land and property from the overflow of the Mississippi River in time of flood by leveeing as he did to provide for the drainage of his land by ditches from surface water in time of excessive rains. Both were natural necessities for cultivation.

Before the year 1850, however, owing to the unorganized efforts of individual planters, there existed only a very imperfect and interrupted system of levees along the front of the Lower St. Francis Basin. Only the higher lands of that small part of the front which was in cultivation were leveed, and the bayous and low ground were open to admit the overflow from the river into the basin to submerge the lower territory lying back from the river front, which, as a consequence, was uninhabited. The first organized efforts towards constructing levees along the Lower St. Francis front were made after the passage by Congress, in 1850, of the law donating the swamp lands to the States for levee and drainage purposes. A board of swamp land commissioners was organized and a system of leyees was constructed, using and absorbing, where possible, the previously constructed private levees. The system, however, was never continuous or complete, but afforded protection to the greater part of the territory against medium floods. The bayous and lowest places were generally unleveed, in all aggregating about 25 miles in length.

In 1858 an extreme flood broke the existing levees in many places, and the system had not recovered from this and the succeeding floods at the outbreak of the Civil War, after which it suffered practically an abandonment for many years. Prior to 1893, at which time the present organization was formed, some more or less organized efforts were made by the separate front counties to maintain the levees, but with poor success.

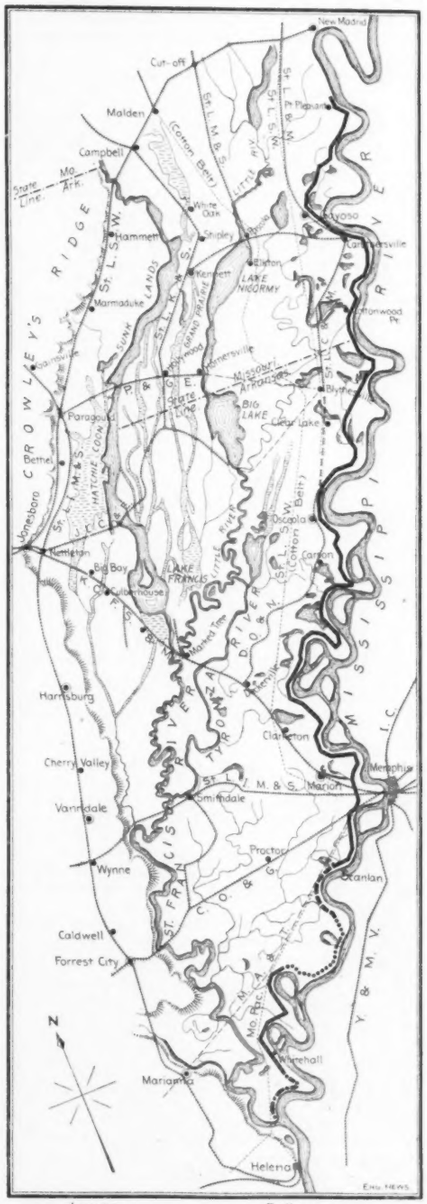
The present State organizations are the St. Francis Levee District of Arkansas, composed of eight counties, or parts of counties, and the St. Francis Levee District of Missouri, composed of three counties, or parts of counties, and are independent of each other, except that each has the same object in view and each depends on the levees of the other for protection. Each district derives its revenues for levee purposes from a tax of 2% on the betterment which will accrue to the area protected. The betterment has been assessed at $2 per acre in every case, both for cultivated and wild lands, thus providing a revenue of 4 cts. per acre. Railroad, town and other property is assessed and taxed according to the betterment. In addition, the State conveyed to the Arkansas board all State lands within the district, including those donated to the State by the United States Government for levee and drainage purposes. In this way and in other ways the district has come into possession of about 200,000 acres of wild land, which has mostly been disposed of at from $1 to $2.50 per acre.

The State of Missouri, prior to the organization of that district, had donated the State lands in that district to railroads under drainage contracts, which were not satisfactorily performed, and the counties interested recently recovered part of the lands, and have used them in constructing canals and for drainage purposes.

The Arkansas district, by a bond issue of $750,000 and further issues of certificates of indebtedness, and the Missouri district, by issuing the latter, have constantly anticipated their revenues for many years and taxed the credit of the districts to a great extent.

The St. Francis Levee Board of Missouri, in conjunction with the United States Government, has constructed a continuous levee from below New Madrid to the Missouri-Arkansas State line, a distance of 47 miles, to a grade only as high as the probable future height of the maximum floods.

The St. Francis Levee Board of Arkansas, in conjunction with the United States Government, connects with the Missouri levee at the State line, and has constructed a continuous levee of about the same grade to Scanlan's Landing, below Memphis, a distance of about 110 miles. A gap of 26 miles, 11 miles of which is now under construction, exists from Scanlan's to Bledsoe. It is proposed to construct the remaining 15 miles of this gap during next year. From Bledsoe a levee has been built to Walnut Bend Landing, a distance of 17 miles. It is proposed to extend the levee from Walnut Bend Landing to the mouth of the St. Francis River, a distance of 12 miles. This will give a continuous line of levee of 212 miles in length, and afford protection to the Lower St. Francis Basin. The expenditures of the two boards together to date amount to about $1,500,000, and the United States Government, in conjunction, has expended about $750,000. The cost per cubic yard has varied from 10 cts. to 26 cts. The material out of which the levee is constructed is the pure soil obtained, usually, immediately in front (river side) of the levee, and varies as greatly as the variety of the soil. The natural surface is depended on as a foundation for the levees, and only in crossing bayous or



Map of the St. Francis Basin, in Missouri and Arkansas.

old lake beds has excessive settling of the foundation occurred. No false foundation has been provided in such cases, but pure earth has been added until the foundation reaches stability by reason of the displacement of the unstable soil by the pressure. The bond between the soil of the foundation and the new earth composing the levee is an important factor in levee construction. The best levee that can be constructed of earth is one of entire homogeneity.

The specifications under which the levees are constructed are practically the same as those used for constructing levees throughout the entire extent of the Lower Mississippi River from Cairo to the Gulf. The general dimensions for the average levee are a crown of 8 ft., with 3 to 1 slopes on both river and land side, which is generally known as the "standard levee." It is expected that such a levee will withstand a flood of long duration and of a height of 2 ft. below the crown of levee. A "topping" of 1 or 2 ft. in height is generally added to crown of levee. These standard dimensions vary with the conditions encountered, and on high embankments the slope is decreased. Banquettes (both straight and broken), with 20-ft. crown, are provided and the crown of the levee is widened.

The funds which have been available for constructing these levees have been limited, and the ultimate grade and strength which will be necessary has not been reached. The United States Government, in assisting in levee construction, has had as an object the confinement and control of the flood water as a means of channel improvement and also the protection to the basin against overflow. For the first purpose it would have been best in locating the levees to have followed more closely around the bends and points than has been done. The bends have been followed more closely than the points, which have, with few exceptions, been thrown out to the overflow of the river by the construction of the levees across their necks. The overflow, as a consequence, has free passage across these points, and the currents are undirected, to the detriment of the channel of the river in those vicinities. The object of the levee districts being first to provide protection to their territory against overflow, they have, without great regard to the improvement of the channel of the river, sought to obtain their object as cheaply and as quickly as possible by the locations which have been made. The general policy which has been pursued by the levee districts in expending their available funds in the construction of new levees, and that which has been pursued by the United States Government in repairing and enlarging levees already built, has left the location of the levees usually to the levee districts, with results stated. The overflow across points which have been thrown out will in time raise them by deposit, and when the control and improvement of the Mississippi River is further developed and sanctioned, and the alluvial lands become more valuable for agricultural purposes, the levees will probably be extended around these points. Some of the considerations which have governed the levee boards in the location of levees are the protection of as much land of the taxpayers as possible; the value of the land so protected and the taxes derived therefrom; the obtaining of as short a levee line on as high ground at as small a cost as possible; a safe distance from caving banks; a protection by foreshore or timber from currents, storms and wind-wash; the drainage of those basins and streams which naturally drain into the Mississippi and not into the St. Francis; the control and direction of the flood water for channel improvement and bank protection, and the regulation of the flood surface slope in the vicinity as affecting the levees and river, and the right of way for levee purposes, which, when not obtained gratuitously, must be condemned at a cost of from $10 to $50 per acre.

An interesting problem in excluding the overflow of the Mississippi River from the Lower St. Francis Basin has been and is the flood heights which will be reached in certain vicinities and along the whole front and on the whole lower river as a consequence. At the head and at the foot of the basin (Cairo and Helena) pools of the river extend, while along the front a shoal extends, culminating about the middle of the front at Cottonwood Point and Fulton. This shoal has been built up in time by the almost annual overflows of the river into the basin. As a result, the variation between high and low water at Cairo without levees has been about 53 ft; at Cottonwood Point and Fulton about 36 ft., and at Helena about 49 ft. By the construction of the levees of the Lower St. Francis Basin the high-water mark at Cairo will not be affected; at Cottonwood Point, Fulton and Memphis, along the middle of the front, the flood height will undergo the greatest change, in additional height amounting to as much probably as 6 ft., while at Helena, at the foot of the front, where the return flow of flood-water from the basin has occurred, the raise in flood height will be slight. Helena occupies the same hydrographical position with regard to the Yazoo Basin as Cottonwood Point does to the Lower St. Francis, and the additional height which has been recorded at Helena during recent years has been caused by the levees of the Yazoo Basin, which are now very far advanced toward the ultimate requirements. The effect on the Helena gage of the closure of the White River Basin, which is very narrow and extends from Helena south along the west bank of the river, will be comparatively slight in addition to that which has been already felt. Again, Helena occupies the same hydrographical position to the Lower St. Francis Basin that Vicksburg does to the Yazoo Basin. The Yazoo Basin, having now been leveed for several years, the effect which such levees have had on the Vicksburg gage may be considered relative to the Helena gage, with due allowance for the effect of the Tensas Basin levees on the Vicksburg gage—Vicksburg occupying about the same hydrographical position to the Tensas Basin that Helena does to the Yazoo, or Cottonwood Point does to the St. Francis.

Colonel Suter, Corps of Engineers, U. S. A., then mem-

lime made April 5, 1901, on neat cement paste samples:

	Water added, %.	Heat, F.
Cement in bin 2 mos.; canvas sack 3 mos.	16	None.
Sample as above, with double the water	32	None.
Cement sampled April 4	32	None.
Sample as above, with half the water	16	8.1°
Sacked cement held for shipment	16	9.9°
Sample as above, with double the water	32	None.

If a cement fails to stand the boiling test, it should be "cured" before using; if it is in cloth sacks this can be done by simply storing it a few weeks, but if it is in paper-lined barrels or paper sacks it will be necessary to empty it on to a floor to allow it to aerate. This test is a severe one, and some cements that give good results in practice fail to stand it. The recent report of the Board of Engineers, U. S. A., is quite fair in its recommendation that "This test is proposed as suggestive or discriminative only, except for works of unusual importance; it is not recommended that a cement passing the other tests proposed shall be rejected on the boiling test."

FINENESS OF GRINDING.—Cement manufacturers have advanced faster than the standard specifications have required in the matter of fine grinding; many cements show a fineness of 3 to 5% residue on a 100-mesh screen, and of 25 to 30% on a 200-mesh screen. The German requirements in 1892 were that "Not more than 10% should remain on a sieve of 900 meshes per square centimeter (76 to the lineal inch), in which the diameter of the wire is half the width of the mesh)." The recent report of the U. A. A. Board of Engineers reads, "92% must pass a sieve of No. 40 Stubbs gage, having 10,000 openings per square inch." The report also states that as there is no Portland cement works on the Pacific coast this standard will have to be lowered there to accommodate foreign cements. The fact is that the works with which the writer is connected, are in California and their standard of fineness is 96% pass the 100-mesh and 76% pass the 200-mesh screen. Two years ago we ground it to the fineness called for in the Government specification, and as shown in the table of tests below, but within the past year the finer standard has been adopted. Recently, to meet the requirements of a large railroad contract, three prices were submitted graduated on a basis of 75, 80 and 85% fineness, through a 200-mesh screen. Foreign cements would not come up to this standard, and the contract was awarded the local manufacturer, both on account of price and fineness of grinding.

It is well understood that only the finely-ground portion of cement possesses cementitious value; the following tabulation of some tests made by the writer for the U. S. Geological Survey and reported in "Water Supply and Irrigation Papers No. 33," illustrates the effects of fine grinding on Portland and on Portland sand-cements. Compare Nos. 3 with 5, and 4 with 6. The terms "Butte pearlite" and "San Carlos quartzite" are descriptive of sand ground with the cement in making sand-cement.

SPECIFIC GRAVITY.—The specific gravity of a cement varies with its age and fineness of grinding. It generally lies between 3 and 3.25. Reference to the following tests illustrate this point:

Brand.	Specific gravity.	Per cent. residue on sieve of 50 mesh.	100 mesh.	200 mesh.
Coarse ground, Colton	3.12	0.85	9.37	33.75
Finer grnd; pass 150 msh	3.06	....	....	.....
Aged; coarse ground, Colton	3.09	....	....	.....
Germania	3.07	.47	6.9	28.12
Gillingham	3.06	.80	7.8	29.33

The method of making the specific gravity determination should be mentioned in any specification or report, for the reason that slight discrepancies often occur in the results obtained by different forms of displacement apparatus generally used. Air bubbles and imperfect manipulation may vary the results. Professor Clifford Richardson has discarded these forms of apparatus in favor of the picnometer and chemical balances. The apparatus used by the writer and the method of its operation are as follows:

La Chatelier's Specific Gravity Apparatus consists of a glass flask of 120 cu. cm. capacity. The neck is about 20 cm. long, the bulb a graduated scale of 3 cu. cm. capacity, divided into 0.1 cu. cm. The diameter of the tube is about 9 mm. In operation, fill the flask to the mark just below the 20 cm. bulb with benzine. Gradually introduce portions of the 64 grammes of weighed cement, using a funnel to get the cement down below the 0 mark on the scale; use care to prevent choking. As the level of the benzine rises to the lowest point of the graduations use caution till the first graduation is reached. The volume of the cement then in the flask is 20 cu. cm., and its weight is got by deducting the weight of the cement left from the original 64 grammes taken.*

The U. S. A. Board of Engineers' report recommends that "The specific gravity of the cement, determined from a sample which has been carefully dried, shall be between 3.1 and 3.25." It will be found that if exceedingly fine-ground and well cured cements are required, this specification will have to be lowered slightly to 3. Details of the methods of making the tests for tensile and compressive strength, and for cracking and checking, according to the Austrian, German, French and American specifications, may be found in Jameson's monograph on "Portland Cement."

Results of tests of Portland sand cements.

No.	Material.	Sand.	Portland cement to sand.	Fineness. 50 mesh.	100 mesh.	200 mesh.	Water.	Strength. 7 days.	28 days.
				Per ct.	*Per ct.*	*Per ct.*	*Per ct.*	*Lbs*	*Lbs.*
1	Colton and Butte pearlite, 1 to 1	2	1 to 5	0.00	0.00	0.00	10	80	300
2	Colton and San Carlos quartzite, 1 to 1	2	1 to 5	.00	00	00	10	90	345
3	Colton, regular	2	1 to 2	.42	7.20	33.20	10	170	385
4	do	3	1 to 3	.42	7.20	33.20	10	140	240
5	Colton, fine ground	2	1 to 2	.00	00	(a)	12	370	465
6	do	3	1 to 3	.00	.00	(a)	12	170	260
7	Colton and Butte pearlite, 1 to 1	3	1 to 7	.00	00	00	10	75	155
8	Colton and San Carlos quartzite, 1 to 1	3	1 to 7	.00	.00	.00	10	55	185
9	Colton, regular	0	1 to 0	.42	7.20	33.20	18	615	660
10	Imported	0	1 to 0	.80	7.80	29.33	18	345	525
11	Imported and sand, 1 to 1	2	1 to 5	.00	.00	.00	10	115	190
12	do	2	1 to 5	.00	.00	(a)	10	125	185
13	do	3	1 to 7	.00	.00	(a)	10	55	90
14	do	3	1 to 7	.00	.00	.00	10	90	140
15	Imported, regular	2	1 to 2	.80	7.80	29.33	10	210	270
16	do	3	1 to 3	.80	7.80	29.33	10	120	175

a Some left on 200-mesh screen.

THE ST. FRANCIS LEVEE DISTRICTS OF ARKANSAS AND MISSOURI.†

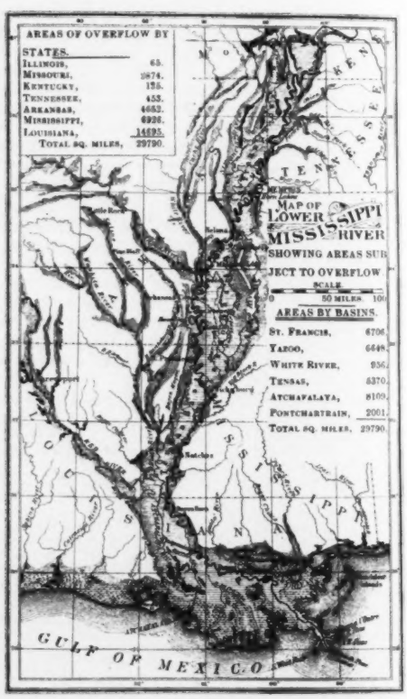
By Harry N. Pharr,‡ Assoc. M. Am. Soc. C. E.

The St. Francis Levee Districts of Arkansas and Missouri compose the Lower St. Francis Basin, which is that territory lying toward the head of the alluvial Mississippi Valley along the west bank of the Mississippi River, and extending from New Madrid, Missouri, at the foot of Big and Little Prairie Ridges on the north, to the mouth of the St. Francis River just above Helena, Ark., on the south, a distance by river of 235 miles, and extending on the west to the uplands of Bloomfield and Crowley's Ridges, having a maximum width of about 45 miles.

It is a territory which without levees is subject to overflow, either directly or indirectly, by the flood waters of the Mississippi River; and comprises an area of about 3,500 square miles. Little and Big Prairie Ridges, averaging about five miles in width and from 10 to 20 ft. above high water, extends from New Madrid, Missouri, on Mississippi River slightly west of a northerly direction Commerce Bluffs at the head of the alluvial valley, with the exception of one or two unimportant depressi forms a barrier to the ingress of flood water to the L St. Francis Basin above New Madrid. The leveeing the Upper St. Francis Basin, now in progress, will p vent the passage of flood water into the Lower St. Fra Basin, even through the few unimportant depressi above mentioned.

In general topography this territory is a great pl sloping southwardly at the rate of about 6 ins. per m and also sloping westwardly at about the same ra Along its western boundary, near the uplands of Cr ley's Ridge, the St. Francis River extends throughout whole length. The general drainage is in a southwe erly direction by the Castor, Little Tyronza and ot rivers, and by innumerable creeks, bayous and slou towards and into the St. Francis River to its confluen at the lower end of the basin with the Mississippi Riv Between the rivers and drainage channels extend ridg some of which were considered above overflow from max mum floods. Lying in about the center of the basin is low area of about 30 miles in length and 20 miles width, known as the Sunk Lands, and said to have bee depressed by the heavy earthquakes known to have o curred in this territory in 1811 and 1812.

Shallow, broad lakes and sloughs cut up this area and allow the spread of the St. Francis River, which traverse it. Artificial drainage and protection will here be neces sary. The convergence of these Sunk Lands at their lower end with the St. Francis River occurs, however below which point the St. Francis River does not over flow from its own waters.



Map of the Lower Mississippi River, Showing the Areas Subject to Overflow.

Slight exceptions to the general drainage of the basin exist. Humphreys and Abbott, in their report on the Physics and Hydraulics of the Mississippi River, in 1861, say:

These north and south ridges, together with the southwest course of the Mississippi, cause several bayous to discharge their drainage, when the swamps are full during floods, directly into that river instead of into the St. Francis. Among such bayous may be named James Bayou, near Island Eight; Bayou St. John, at New Madrid; Walker's Bayou, near Island Fifteen; Mill Bayou, opposite Island Thirty; Wappowocca Bayou, near Island Forty, and a bayou near the head of Island Forty-six (Four Mile).

Some artificial system of drainage for the local basins of these bayous will have to be devised before the continuous chain of levees upon the bank of the Mississippi, so necessary to reclaim the swamp lands, is possible. In 1858 many levees, especially in the vicinity of the mouths of these bayous, were washed away by crevasse water pouring back into the Mississippi.

Without going too far back and into the detail of the geological formation of the alluvial basins of the Mississippi Valley, it is well to state that the Lower St. Francis Basin is composed of a rich loamy soil of great fertility, having been built up by the silt-bearing overflows of the Mississippi River. The character of the soil differs in localities, varying from black buskshot to sandy

*Specific gravity is the ratio of a certain volume of the substance to an equal volume of water. The gramme is equal in weight to 1 cu. cm. of water weighed in vacuo, at its maximum density, or — 4° C.

†A paper read before the Memphis Engineering Society, Sept. 10, 1901.

‡Chief Engineer St. Francis Levee Board and Vice-President Memphis Engineering Society, 49 Southern Express Building, Memphis, Tenn.

ways were spaced at distances apart varying from 3 ft. at the highest part of the wall to about 10 ft. at the lowest part. Altogether 30 ways were used for each wall. The actual moving of the wall was accomplished by means of screw jacks. Thirty of these were used for each wall, and they were placed in a trench cut in the approach floor at points exactly opposite the 30 track ways. The feet of the jacks abutted against the concrete floor and their heads against the wall. As the jacks were extended and the wall moved forward they were followed up with blocking. By means of these jacks the wall was pushed gradually ahead. Neither wall was cracked or injured during the moving process.

The second part of the work was the widening of the concrete and steel subway structure. This structure was about 275 ft. long and was made up of steel bents spaced 5 ft. apart and filled in with concrete, as shown by Figs. 1 and 2. It will be noted that an interior row of columns separated the two tracks. The foot of each wall column and of each interior column was secured to the concrete column foundation by anchor bolts. Around the whole structure, as is shown by the illustrations, a layer of water-proofing made of tar paper and asphalt was placed. On the side walls this water-proofing was held in position by a wall made of a single course of bricks held in place by the earth filling.

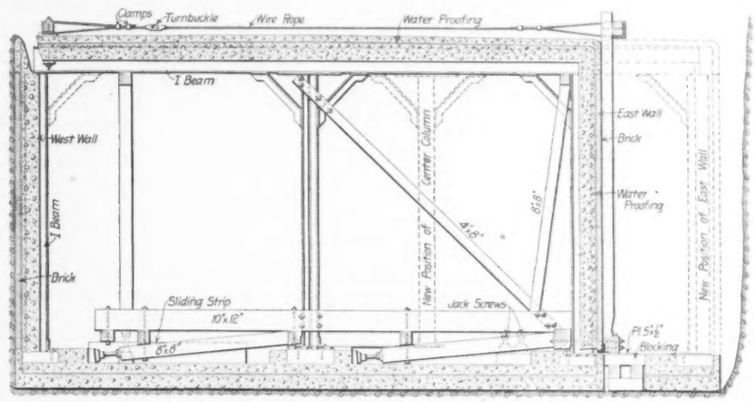


Fig.1. Subway ready for Moving of East Wall and Roof.

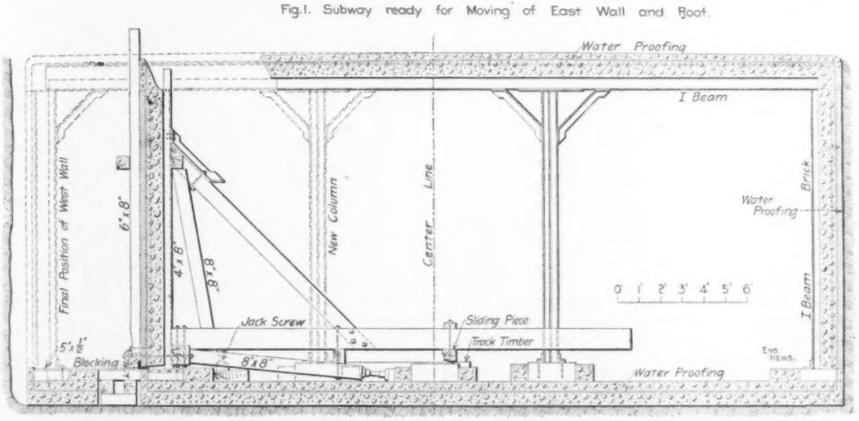


Fig.2. Showing East Wall and Roof Moved; West Wall ready for Moving.

FIGS. 1 AND 2. TRANSVERSE SECTIONS OF RAPID TRANSIT RAILWAY STRUCTURE BETWEEN 135TH AND 138TH STREETS, SHOWING METHOD OF MOVING SIDE WALLS.

To widen the structure described, the roof was cut away from the top of the west wall, as shown by the illustrations, and the roof, east wall and center columns were moved in one piece a distance of 5½ ft. to the east, while the west wall in one piece was moved the same distance to the west. This left a gap of 11 ft. in the roof. To close this a second row of interior columns was erected, and I-beam roof beams were inserted to complete the four-column bent called for by three-track construction. The estimated total weight of the east wall and roof was about 3,000,000 lbs., and the west wall, which was about 16 ft. thick and 15 ft. high, weighed about 700,000 lbs. So that the structure would not be injured or distorted by the masonry, it was braced as shown by the illustrations. Along the outside and inside of the side wall of the structure continuous longitudinal timbers were laid and bolted together through the wall. Cross timbers and diagonal bracing, as shown, were put in at every alternate bent, or 10 ft. apart. To support the brick wall forming the protection for the water-proofing, the outside longitudinal timber had an angle bolted to it having one leg extended under the brick, and small angle legs were bolted to the top to hold the foot of a vertical timber which was held at the top by a continuous longitudinal timber secured by wire ropes, as shown. These vertical sticks on the outside were spaced 10 ft. apart. The anchor bolts, which secured the feet of the several columns, were cut off flush with the top of the bed plates. Jack screws were then arranged under the west side of the roof, as shown, and the latter was raised sufficiently to lift the center line of columns, so that the base plate was clear of the bolts. Sliding strips of 5 × ½-in. iron were then placed under the bed plates and blocked up so that the bed plates would pass over the bolts. The jack screws were then moved under the east side of the roof, as shown, and the east wall and columns were similarly raised. Slideways were then made of timber blocking to carry the structure to its new location, and the jacks were placed in horizontal positions, as shown. About 60 jack screws were used. The moving of the west wall was accomplished in exactly the same manner as the east wall was moved, except for such differences as are obvious from the drawings.

As previously stated, the entire work described was carried out by the Terry & Tench Construction Co., of New York, and we are indebted to them for the information from which this description has been prepared. L. B. McCabe & Bro. constructed the work originally under a subcontract for sections 13 and 14 of the rapid transit railway.

NOTES ON PEAT-FUEL, IN SCANDINAVIA.

It is estimated that the possible yield of the Swedish peat bogs is equivalent to 3,000,000,000 tons of coal; and the high price of coal proper in Sweden is calling renewed attention to these fuel deposits. In a paper recently presented to the Institution of Civil Engineers, Mr. Alfred Bache, Assoc. Inst. C. E., presents the substance of thirteen separate articles on peat-fuel which have lately appeared in the technical papers of Sweden, Norway and Denmark. From this summary the following points of general interest are taken:

At a late meeting of the Swedish Peat Cultivation Society, Mr. Alfred Larsson spoke of the best methods of converting peat into good fuel. In 1896 over 1,000,000 tons of peat-fuel were manufactured, and chiefly used for metallurgical purposes in the midlands of Sweden. This peat had a heating power in the ratio of 1 to 1.8, as compared with coal costing $4.00; while the peat cost $2.22 per ton. The actual cost of manufacturing the peat-fuel was 80 cts. to $1.60 per ton, and the transportation charges were the same for peat as for coal.

Mr. Larsson thought the most efficient method of manufacturing peat-fuel was that pursued at Haarlem, in Holland. The bogs are there covered with 12 to 20 ins. of sand and clay, and the peat bed is from 5 to 6½ ft. deep. The sand and clay are used to raise the level of the drying area; or, in making dikes about the peat working. The bog is leased with the understanding that the peat must be removed and the land drained dry within 30 years; and the rental of the bog amounts to at least 72 cts. per ton of peat. The peaty mud is scooped up by an excavator carried by a large iron scow, which will float in about 12 ins. of water. The peat is kneaded on this scow and is delivered to the drying floor, and spread in layers 2 ft. thick, where it dries down to half its thickness in a couple of days, and is twice trodden on this bed. It is then cut into blocks 3 ins. square and 5 ins. long and stacked on end to dry. Twelve of the excavators described turn out 50,000 tons of dry peat per year, and deliver the kneaded peat to the drying-floor at a total cost of 72 cts. per ton. The dried peat is worth $3.20 to $3.40 per ton at the bog; and is so valued by the lessee in taking payment in peat for work done for the landlord.

Dried peat briquettes have been made in Sweden for ten years directly from peat-mull containing 35 to 60% of water. This mull is crushed between rolls, sifted through a sieve with 0.4-in. mesh and mechanically conveyed to a drying-room, where the water is reduced to 15 to 20%. It is then pressed, at about 176° F., into briquettes having a density of 1.0 to 1.2. These presses produce from 30 to 50 tons in 24 hours; but to make good and durable briquettes the peat must go into the press with 18% of water.

The conversion of peat into charcoal is not commercially successful. Air-dried peat, with 25% of water, does not yield more than 40% of charcoal. Its heating power is 6,300 B. T. U. per lb.; and of this only 5,040 units remain after reduction to charcoal, or a loss of 20% in heating power, apart from the cost of carbonizing. To illustrate the difficulties of treating peat, it is stated that to get one ton of dry peat, four tons of water must be eliminated; and one ton of peat-charcoal represents 2½ tons of dry peat and 10 tons of water. With an air-temperature of 160° F. in the drying-shed, at least 28,000,000 B. T. U. are required to make one ton of peat-charcoal, which can only develop about 22,600,000 B. T. U. To this loss must be added the cost of power for supplying the current of drying air and the heat expended in carbonizing the peat.

Prof. N. Steenberg, in discussing this loss of combustible matter by coking, says that besides the water and carbonic acid driven off in this process, some tar is formed, smoke becomes visible and the condensed water always contains a little acetic acid and wood-alcohol; carbonic oxide and a little marsh-gas also escape. He says that while peat fuel yields an excellent gas for lighting and heating, or for a household fuel, it is too largely mixed with carbonic acid and steam to be suitable for gas-engines.

ber of the Mississippi River Commission, and President of the Missouri River Commission, in the report of the Mississippi River Commission, First Session, Fifty-fourth Congress, Appendix L, said:

In a general way it has been shown that the large increase of flood-height has been mainly confined to intermediate points along the great swamp basins. At the lower end of these basins it has been slight, as for instance at Vicksburg. The only feature of uncertainty which now remains is as to the effect of leveeing the St. Francis Basin. That this will increase flood-heights at Memphis and all points above there seems certain, but so far as concerns the river below Helena there is this much to be said: All the water which reaches the main stream above the junction of White River now passes Helena between levees, and hence has probably produced its maximum effect.

Humphreys & Abbott in their report on the Physics and Hydraulics of the Mississippi River, Chapter 6, say:

This general conclusion as to the effect—uncorrected by the reservoir influence of the channel—exerted by the St. Francis bottom upon the high-water discharge at Helena will be compared with the corresponding effect of the Yazoo swamps upon the discharge at Vicksburg, which, as already seen, was accurately determined. These two swamps are similar in dimensions, and, usually, in the depth of overflow, and general conclusions based upon the analogy existing between them are entitled to some confidence. . . . The following final conclusions respecting these swamp regions in their unleveed condition must therefore be considered established: First, they produce no effect whatever upon the volume of the maximum discharge of the Mississippi, above and below them, in great flood years. Second, they did reduce this volume along their fronts, and by an amount which increased from their upper to their lower limits. Third, they retarded both the rising and the falling of the river at all points below them. Fourth, they tended to increase the duration of the floods throughout the alluvial region.

Relative to the high water of 1858 they report that "the swamp could not have acted as a non-returning reservoir, but must have discharged a much larger volume back to the Mississippi."

It is by doing away with the return flow of flood-water to the Mississippi River from the Lower St. Francis Basin at its lower end near Helena by confining the floods in the Mississippi River by levees along the Lower St. Francis Front and passing them on to the Gulf as they come, that the gage at Helena is not expected to experience any increase in flood heights.

The pool, shoal and pool regime of the Mississippi River governs to a great extent the slopes, the velocity of flow and discharge. At stages below bank full or stage of overflow, a 12-ft. rise at Cairo gives about a 9-ft. rise at Memphis and about a 12-ft. rise at Helena. This relation is modified by the rapidity of the rise and its consequent variation in rate of elongation; 40 ft. at Cairo gives about 30 ft. at Memphis and 40 ft. at Helena, which are about the respective bank-full stages at those points. It might be expected that the relation which these gages have held below these stages would be maintained above them when the river is confined by levees and the bank line raised or the outflow prohibited. Such might be the case if it were not for the fact that the regime of the river is changed at that stage, and the relations which have been built up naturally by the river itself below bank-full stage no longer exist. For at this stage the Helena pool has filled, tending to form a uniform slope and flow with the shoal above and below, and at higher stages it passes and delivers the water over the shoal below as fast as it comes over the shoal above, and thereafter a 12-ft. rise at Cairo gives about a 9-ft. rise at Memphis, and about a 9-ft. rise at Helena. It is therefore expected that when the levees are completed and all the overflow water confined, that a 52-ft. stage at Cairo will give about a 41-ft. stage at Memphis, and about a 51-ft. stage at Helena. This stage was exceeded at Helena in the 1897 flood, the gage there having reached a reading of 51.8, and the river was still rising when the Mississippi levees opposite the lower end of the St. Francis Basin broke and admitted the flood into the Yazoo Basin. The Lower St. Francis Basin, however, at that time was practically unleveed, the old state levees, where utilized, having broken in many places, freely admitting that great flood and consequently allowing an excessive return flow at the foot of the basin, thus causing the abnormally high stage which occurred. Had the Lower St. Francis Basin been leveed against that flood it is probable that 51 ft. would not have been exceeded on the Helena gage.

Levee grades have been established 2 ft. above the foregoing provisional high-water stages as that which will be ultimately necessary to give a safe degree of security to the basin against overflow at an economical cost, the grades at intermediate points being determined by their location, the slope of the flood surface and the pond or draw of the river in their vicinity. All elevations are referred to the Memphis datum which is established at 190.84 ft. below the zero of the Memphis gage.

The drainage of those basins lying behind the levee and draining into the river, made necessary by the leveeing of their outlets, has been or is proposed to be diverted toward the rear of the basin into the tributaries of the St. Francis River. This has been done by the construction of canals, generally of an 8-ft. 3 to 1 slope cut across the ridges surrounding the basins. These canals seldom exceed five miles in length, and have been constructed at a reasonable cost to drain the basin to within the banks of their outlets. It is proposed to give such further relief as may become necessary by the use of siphons over the levee at these outlets. Culverts through the levee for drainage are considered dangerous and are not used. They have been practically abandoned in the construction of levees on the lower Mississippi River.

The levee districts have adopted one of the two ways of obtaining protection against the overflows of the Mississippi River, of either preventing them or providing against them. They have adopted the latter, the only recourse within their own jurisdiction. To prevent the overflows of the lower Mississippi River by diverting its tributaries—such as the Upper Missouri into the Red River of the North or the Tennessee River into the Tombigbee or Alabama River is impracticable. To divert the overflow of the Mississippi River at the head of the St. Francis Basin into Black River, Cache River and St. Francis River is impracticable as to cost and would only diminish the depth of overflow in the St. Francis Basin, while providing more territory to be submerged, and only to have their waters again returned to the Mississippi. To create reservoirs of significance for the protection of the St. Francis Basin on the upper rivers has been found impracticable, and prohibitive. Places for the construction of outlets of sufficient importance to afford any relief to flood heights do not exist along the St. Francis Front, and their construction at any point where possible on the river below the St. Francis Basin would not, even temporarily, materially reduce flood heights for any considerable distance either above or below their vicinity.

The cutting off of points and straightening of the river channel reduces the height of floods above and increases the height of floods below them for short distances, and is of no material benefit to flood heights, even temporarily, but is very detrimental to the regime of the river and the establishing and fixing of levees.

The river flowing through its own alluvium must fix its slopes and establish its own regime. When it has nearly done so it may be fully controlled by artificial assistance, and the plan pursued generally is to assist and hold the river where it is without directing or allowing any changes.

The construction of levees along the lower St. Francis Front is so recent that no observations have been made as to their effect on the fill or scour of the river bed. On the lower river, however, where levees have existed for a longer time, and accurate observations have been made no satisfactory conclusions as to whether filling or scouring has taken place have been reached. It is reasonable to consider, however, that the cause (the overflow into the basin) which has produced a pool at each end of the St. Francis Basin and a shoal along its front, when confined to the river, will in time tend to regulate the depth by scouring on the shoals and filling the pools, which will afford a more satisfactory condition of the river channel. Whether or not the general bed of the river fills or scours may take centuries to determine.

The plan which has been proposed to afford relief from floods to the lower Mississippi Valley by converting the St. Francis Basin into a reservoir is possible, but is impracticable and unnecessary. It would cost more to so convert it than to complete all the levee systems including its own from Cairo to the Gulf of Mexico. With it converted into such a reservoir levees would still be necessary below, and danger from breaks in the reservoir during overflow would greatly increase the danger from floods.

Humphreys & Abbott, in their report mentioned in their article on reservoirs, conclude as follows:

To guard against misconception it may be well to repeat that the advantages of a reservoir system upon certain Western rivers, for certain objects, are not questioned. By it the low-water navigation of important streams flowing into the Ohio—perhaps of that river itself and possibly even of the Mississippi—may be improved. The data for deciding whether the advantages accruing would be commensurate with the expense of constructing them have not yet been collected. But the idea that the Mississippi Delta may be economically secured against inundation by such dams has been conclusively proved by the operations of this survey to be in the highest degree chimerical.

No deep waterway or ship canal through a reservoir so formed as has been suggested is practicable, but if levees are built and maintained throughout the length of the river it does not seem unreasonable to prophesy that when the shoals have scoured and the pools filled and uniform depths obtained, ocean-going vessels will land at Cairo.

After the deplorable suggestion of converting that alluvial district into a reservoir, the improvements and developments which have taken place in the basin in the last eight years, since the levees have been rapidly constructed and protection afforded, may be more happily mentioned. The population has about doubled; the area of cultivated land, then only about 10%, has increased about 50%, the value of wild lands has on an average increased about 200%, and cultivated land about 50%, their respective values being now on an average about $6 and $60 per acre; the lumber and timber industry has increased about 400%; ten new lines of railroad, aggregating nearly 300 miles in length, literally gridironing the basin, have been built, and others aggregating as much mileage within the basin have been projected and proposed.

When we consider the work now in progress on the Nile River in constructing dams at Assouan and Assiout and other work there, estimated to cost $25,000,000, in order to add 2,500 square miles to the cultivable area of Egypt, the value of which it is estimated will amount to $400,000,000, we may realize to what the addition of 3,500 square miles of the alluvial St. Francis Basin, not exceeded in fertility by the Valley of the Nile, will in time amount. This it is estimated can be obtained and maintained by levees at an ultimate cost of less than $5,000,000, with the present strong organizations and the assistance of the general government assured, and the front almost entirely closed with a serviceable levee, the reclamation of the Lower St. Francis Basin is more than half accomplished.

WIDENING A SECTION OF THE NEW YORK RAPID TRANSIT RAILWAY BY MOVING THE SIDE WALLS.

That portion of the Rapid Transit Railway structure between 135th St. and 138th St., on Broadway, in New York city, was originally designed to be a two-track subway, built of I-beams and concrete, with an open approach from the south formed by two brick-faced concrete retaining walls. The structure was completed about a year ago, and was of the standard construction adopted by the Rapid Transit Commission. After its completion, however, it was decided that a three-track subway was necessary for this section, and, consequently, that the subway and approach already completed would have to be widened at least 11 ft. to provide room for the additional track. It was at first expected that the only way to accomplish this widening would be to tear down the structure already built and to rebuild the section anew. The Terry & Tench Construction Co., of New York city, however, submitted a proposition to move apart the original side walls, which was accepted. The method proposed was to move back each of the brick-faced concrete retaining walls of the open cut approach and set them on new foundations, and, for the subway portion, to move the roof and one side wall one way and the remaining side wall the other way, and to set them on new foundations, and, finally, to fill in the gap in the roof.

The portion of the work which was first attempted was the moving of the approach walls. These walls were each 200 ft. long and ranged in height from 4 ft. to 14 ft. In side elevation they were wedge-shaped, since their bottoms followed the descending grade of the approach track and their tops rose on an ascending grade to the roof of the subway. The walls varied from 4 ft. to 9 ft. in width on the bottom, and were 2½ ft. wide throughout on the top. The faces of the walls were vertical, and their backs were stepped. Under each wall and extending clear across the bottom of the approach cut there was a concrete footing about 14 ins. thick.

The method adopted for moving the walls to their new location was as follows: The concrete footing or floor was cut through by a trench about 30 ins. wide parallel to each wall and close to its vertical face. The ground was then excavated back of the walls so as to give the increased width of cut. From the back of each wall 12 × 12-in. timbers were then thrust endwise under the footing. These timbers were just long enough in each case to project slightly beyond the front and rear faces of the footing. From their rear ends concrete ways or walls were carried back to and beyond the foundation site of the wall when in its new location. Lengthwise of the 12 × 12-in. timbers were placed 5 × ½-in. strips of steel long enough to reach the full length of the concrete ways. By means of wedges and blocking the 12 × 12-in. timbers were raised a sufficient height to lift the wall clear of its foundation bed. When in this position the top of each 12 × 12-in. beam was just flush with the top of its corresponding concrete way, and by means of the steel strip a perfectly level and continuous track therefore existed from the front face of the old wall to the rear face of the new foundation. Upon these parallel tracks the wall was to be slid into its new position. To give the wall a smooth bearing on the tracks, steel plates were embedded in the concrete footing so as to bear upon the track strips. A new footing for the wall was then constructed in the spaces between the concrete ways, but its top was kept a few inches below the tops of the ways so that any irregularities on the bottom of the wall would clear the new footing when the wall was being slid into position over it. Afterwards these spaces were filled in with heavy gravel grout. The track timbers and the concrete

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CHARLES WHITING BAKER, SECRETARY AND MANAGING EDITOR.
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There is a piece of engineering work now in progress in the United States which, when completed, will make available for settlement and cultivation a tract of land as fertile as any on the globe, and larger than the combined states of Rhode Island and Delaware. The work we refer to is the protection by levees of that part of the states of Arkansas and Missouri known as the St. Francis Basin. We doubt if there is anywhere in the world any piece of engineering work in the department of land reclamation which promises to accomplish such great results for so slight expenditure; and yet it is a safe guess that not one person in a hundred in the United States ever heard of the St. Francis Basin.

On another page of this issue we publish an interesting account of this work, written by the engineer in charge; and it may be of interest to make some comparisons to show the importance and magnitude of the work.

As most of our readers know, the alluvial lands of the Lower Mississippi, of which the St. Francis Basin forms a part, are unsurpassed for productiveness, and the area of waste land is very small indeed. Nearly the whole of this great tract, some 3,500 sq. miles in extent, has been subject to overflow by the Mississippi, and has thus been left in a wild and uncultivated state from time immemorial. With the floods shut out by levees, however, nearly all this great territory can be brought under a high state of cultivation, and only by comparison can we form an estimate of the great benefit to the nation and to humanity at large in adding this fertile agricultural land to the area available for homes and farms.

As stated above, the St. Francis Basin comprises about 3,500 sq. miles, and the estimated ultimate cost of protecting this from overflow by permanent levees is only some $5,000,000.

Every schoolboy knows of the work Holland has done in reclaiming lands from the sea. In another article in this issue we describe Holland's latest project for reclaiming the Zuyder Zee, which has been under consideration for many years, and is now postponed on account of financial difficulties. As shown in the article, the Zuyder Zee project would reclaim 750 sq. miles of land, and its total cost would reach the enormous sum of nearly $79,000,000, while the work would extend over 33 years.

Another interesting comparison is made by Mr. Pharr with England's work in Egypt. By building the Nile dams at Assouan and Assiout and other necessary works, at a cost of some $25,000,000, about 2,500 sq. miles of the Nile valley will be added to the territory available for cultivation. In other words, the Egyptian reclamation will cost $10,000 per sq. mile. The reclamation of the St. Francis Basin will cost only $1,430 per sq. mile.

Mr. Pharr, in his paper, states that improved land in the St. Francis Basin is already worth $60 per acre. It is a safe assumption that a few years hence will see this value raised to $100 per acre. If we allow one-fourth the area of the basin to be waste land, we reach a total valuation for the entire district, when land values reach the point just named, of no less than $168,000,000. Surely one will go far to find an engineering enterprise which promises so large and certain a return for so small an expenditure.

It seems strange indeed that the granting of such a franchise should now be under serious consideration at East Orange, N. J., after all that has been said and written against the granting of perpetual franchises. It is true that the circumstances are somewhat unusual, in that the United Electric Co., which is seeking the franchise, offers to place some of its wires underground, and besides furnishing seven 1,200-c. p. arc street lights free, to light the city hall and police station without charge, and to supply other municipal lighting at 3 cts. per K-W. hour less than the rates to private consumers. Besides this it offers to have a readjustment of the compensation for the franchise made at the end of each 25 years.

Notwithstanding all these liberal provisions, we cannot see that the grant of a perpetual franchise is justifiable. What moral right has the present generation to bind all future ones by a contract of this sort? We are manifestly too ignorant of the conditions which may exist a half-century or a century, or two centuries hence to safely enter into contracts to bind those who will then be living. The law recognizes this already, to a certain extent. The maker of a will, for example, cannot control the disposition of property bequeathed to persons beyond one or two generations. Why should not a similar restriction be made in the disposition of public property? If it is wise to prevent a man from dictating the disposition of his own property beyond a certain period after his death, how much more would it be wise to restrict public officials from entering into contracts to bind all succeeding generations?

The collapse of the National Asphalt Co. is a sharp reminder to its promotors—and, unfortunately, to many innocent investors in its securities—that a trust with a supposed great monopoly as its only asset cannot succeed in this country. Instead of proposing to cheapen production by a wiser administration and the cutting down of unnecessary expenses by a consolidation of departments, this combination of companies attempted to create a monopoly, and by killing competition, to enhance the cost of paving throughout the country. As one editor expresses it: This company proposed in some mysterious and unexplained way, to so manage its affairs that the separate companies, earning 4% on $10,000,000, would by mere consolidation be enabled to pay 5% on $30,000,000. This could only be done by raising prices to its customers to an abnormal extent; and the proposition invited the collapse that so speedily followed. In the consolidation the services of many able men of long experience in just this class of work were dispensed with; and with an artificial enhancement of values their opportunity presented itself for competition at businesslike figures. The monopoly expected did not materialize. Contracts in one large city after another were gained by the competitors of the trust, and the bubble burst. As a lesson in economics and a warning to the organizers of similar consolidations the collapse of this concern will work some small measure of benefit to partially compensate for the loss suffered by too-innocent investors. Consolidations of this class have justified the public demand for a general reform in American methods of company organization, and for such publicity in the matter of assets, financial results of operations and methods of conducting business as will protect, in some measure, the public from being too easily beguiled with fulsome promises.

A most peculiar plan for the improvement of the New York canals is recommended by Governor Odell in his message to the legislature. It is in substance that new locks be built on the present Erie Canal to accommodate barges of 1,000 tons capacity and that the canal prism between the locks be deepened to 9 ft., all at an estimated cost of about $28,700,000. The Governor's idea, apparently, is that eventually the whole canal will be enlarged to accommodate 1,000-ton barges and that with locks of this capacity built as an "entering wedge" the taxpayers will later be induced to vote the money to enlarge the entire canal.

A more wasteful and absurd plan of carrying out a piece of public work it would be difficult to conceive. In the first place, if the State is to build a canal for 1,000-ton barges, it ought by all means to abandon a large part of the present Erie canal and adopt one of the improved routes surveyed by the engineers a year and a half ago. To attempt to build a canal for 1,000-ton barges on the line of the present Erie canal will result in an enormous and unnecessary expenditure of money, and the canal when completed will be far more expensive to maintain and far less advantageous for navigation than a canal located on a better route. To put the contrast with reference to additional cost into figures: The engineers' estimate of the cost of a 1,000-ton barge canal on the present Erie canal route modified was $87,317,570. By the more favorable route through the Mohawk and Oswego Rivers the estimate was only $54,708,279.

Even if it were decided to adopt the present Erie canal route for a 1,000-ton barge canal, it would be folly to first spend $13,694,000, as the Governor recommends, to make a 9-ft. channel on this route, for the bulk of this expenditure would be a total loss when it came to constructing a larger channel.

The Governor appears to think that the canal is a mere ditch, which can be deepened to 9 ft. at present and then gradually deepened and enlarged to accommodate boats of greater draft and beam as time goes on. But the canal is not a ditch. It is a structure, built to carry water, with its sides and bottom protected by puddle from percolation and its slopes paved to protect from wave action. To enlarge the channel from 9 ft. to 12 ft. would require the demolition of most of the structure prepared for the 9-ft. waterway.

Fortunately the chances appear almost infinitesimal that the Governor's scheme for canal improvement can be carried into effect. Even if it should receive the approval of the legislature, it would have to be later approved by popular vote at a State election, and the chances of securing a popular vote for the expenditure of nearly $30,000,000 on the canals, admittedly as an entering wedge for the expenditure of more than twice as much more, would almost certainly be turned down by the voting taxpayers.

The advocacy of water meters by the governor of a State in a formal message is something new, but is quite in keeping with the spirit of the times and with the reputation for keen business ability and common sense generally accorded to Governor Crane of Massachusetts. The Governor, in his annual message, recommends that the municipalities of the Metropolitan Water District pay for

water in accordance with the quantity consumed, instead of, as now, on the basis of population and assessed valuation. This plan, the Governor thinks, would increase the use of meters on private services, already in vogue in some of the communities, and would greatly curtail waste. There is much to be said in favor of the proposition and sooner or later it seems bound to prevail. Possibly, however, some modification may be necessary to cover the immense outlays for fire protection and other public services, which would not be met by ordinary meter rates on the relatively small quantities of water thus used.

NICARAGUA OR PANAMA.

When we reviewed the report of the Isthmian Canal Commission and its comparison of the Panama and Nicaragua routes for a ship canal in our issue of Dec. 26, we expressed the opinion that the selection by Congress of the Nicaragua route was now as good as settled. During the holiday recess of Congress, however, the Panama stockholders have held a stormy meeting, President Hutin has resigned, and the new officers of the company on Jan. 4 presented a definite offer to dispose of all their rights, properties and franchises on the Isthmus for $40,000,000 in place of the $109,141,500 named by President Hutin.

Under these circumstances the discussion of the relative merits of the two canal routes is being revived in the public prints, and a good deal of information and misinformation is being circulated. That these changed conditions may considerably delay the final action of Congress on the matter is quite probable. In the House, Mr. Hepburn's bill for a Nicaragua canal will probably be passed with little opposition and very likely before this issue of Engineering News reaches its readers; but in the Senate, advocates of the Panama route are reported to be sufficiently numerous to delay any decision in favor of Nicaragua, at least.

Meanwhile, since the matter becomes again one of public interest, we think it worth while to summarize more fully than we have yet done the relative advantages and disadvantages of the two routes, as set forth by the Commission in its report. There appears to be a more or less general impression that the advantages are all on the side of Panama, and that if the Panama company could be bought out for a sum equal to the excess which it will cost to build the Nicaragua canal as compared with the cost of finishing the works at Panama, the United States should by all means choose the Panama route.

To those who hold such an opinion, we recommend careful study of the merits and demerits of the two routes as summarized below. It will be seen that the advantages are by no means all on one side.

ADVANTAGES OF PANAMA ROUTE.

(1) Shorter distance,—49 miles against 183.66 miles for Nicaragua. Of this distance 36.41 miles on the Panama route and 73.78 miles on the other are in standard canal section or in harbors and entrances; 12.68 miles at Panama is in Lake Bohio and 113 miles at Nicaragua are in Lake Nicaragua and the canalized San Juan.

(2) Fewer locks,—five in all against eight at Nicaragua.

(3) Lower summit elevation,—85 ft. against 107 ft. at Nicaragua.

(4) Less curvature,—1,568° and 26.44 miles less, according to the Commission's summary. The alinement of the Panama route is remarkably good. Except one curve of 3,280 ft. radius at the entrance to Colon harbor, where the bottom width is 500 to 800 ft., there are no curves on the canal with a smaller radius than 6,200 ft. and only three with radii shorter than 8,200 ft. On the Nicaragua route there are 10 curves of less than 5,000 ft. radius and 23 of 5,000 to 6,000 ft.

(5) Shorter time of transit,—12 hours for a deep-draft vessel as compared with 33 hours at Nicaragua. This may be taken as the measure of the advantage to vessels navigating the Panama canal which are involved in (1), (2), (3) and (4). It should be noted that this comparison of time of transit was very carefully worked out by the Commission, careful estimates being made of speed on tangents and on curves, in canal sections and in open channels and time required for lockage and for passing other vessels.

(6) Smaller cost of construction,—$144,233,000 to complete the work at Panama as compared with $189,864,000 to build the Nicaragua Canal.

(7) Lower cost of maintenance and operation,—$2,000,000 compared with $3,300,000 for a Nicaragua Canal.

(8) Better harbors.—The Commission has planned for the construction of good harbors for both routes and says that when such harbors are completed one route would have little advantage over the other. "The balance of advantages (as respects harbors) including maintenance and operation, however, is probably in favor of the Panama route."

(9) More prompt beginning of work.—At Panama, harbor and railroad facilities already exist and canal work could go forward as soon as a working force could be organized and machinery assembled. The Commission estimates that one year would be required for this at Panama, while two years would be required at Nicaragua, on account of the lack of harbor and railway facilities at the start.

(10) The Panama Canal would furnish a shorter route for traffic between North Atlantic ports and the west coast of South America; the saving in time over the Nicaragua route being about two days or about one day for vessels sailing from Gulf ports.

ADVANTAGES OF NICARAGUA ROUTE.

(1) Works of less magnitude.—Under the old Menocal plans for the Nicaragua work the Ochoa dam and the great Divide cut were frequently brought forward as arguments against the Nicaragua route. The new location made by the Isthmian Canal Commission, however, involves no works of the magnitude of those at Panama. The Bohio dam at Panama involves more difficulties and will cost a half more than the San Juan River dam proposed at Conchuda on the Nicaragua route. The Culebra cut at Panama still contains 43,000,000 cu. yds. to be removed, while on the Nicaragua route the Divide cut, which is the largest single piece of work, contains only 18,000,000 cu. yds.

(2) Shorter time for construction.—The Commission estimates that the Nicaragua Canal could be completed in eight years, two years of which is allowed for preparation. At Panama, eight years will be required for the Culebra excavation alone, so at least ten years must be allowed for the entire work of construction.

(3) A shorter trade route.—Notwithstanding the longer time required to pass through the Nicaragua Canal, vessels using that route for the voyage between North American Pacific ports and Atlantic ports of the United States or Europe will save about a day over the Panama route. The same advantage exists for traffic between Atlantic ports of the United States and the Orient. For ports on the Gulf the Nicaragua route is nearly two days better than Panama. Inasmuch as it is this traffic more than any other for which the United States is to build the canal, it would appear that this advantage of the Nicaragua route much more than outweighs the advantage of the Panama route for trade to the South American countries west of the Andes. This is one reason, we believe, why the Pacific coast has always been clamorous for a canal at Nicaragua and against one at Panama.

(4) Better for sailing vessels.—It has long been known that the calms in the bay of Panama, and the steady trade winds at Nicaragua make the latter the preferable route for sailing vessels. The Commission thinks that sailing ships are being so rapidly displaced by steamships, however, that the point is of little practical importance. On this, however, we are disposed to disagree with the Commission. It is quite within the possibility, if not the probabilities, that the present century may witness a revival of the use of sailing vessels for moving merchandise over long voyages, such as that across the Pacific. The increasing cost of coal, and the development of improved types of sailing vessel are among the factors which are likely to bring this about.

(5) Local development.—A canal across Nicaragua will result in the development of a considerable territory in Nicaragua and Costa Rica which will furnish trade to the United States. In Panama no particular development is to be looked for.

(6) Better hygienic advantages.—This important matter is on the side of Nicaragua. "The notorious mortality which attended the construction of the Panama Railroad and later the operation of the Panama Canal Co. has taught a lesson which will not soon be forgotten for that route," says the Commission. At Nicaragua, the experience of the old Maritime Canal Co. was very favorable. The present Isthmian Commission had 20 working parties in Nicaragua, with 159 engineers and other assistants and 455 laborers. Among the white men of this force, "there were fewer cases of sickness than would probably have occurred among the same number of men employed in some parts of the United States." Undoubtedly, as the Commission says, conditions at Panama are more likely to cause epidemics among the forces employed on the canal than at Nicaragua.

The above list comprises all the advantages and disadvantages of the two routes presented in the Commission's reports (with one important exception). It will be seen that the Panama route offers undoubtedly the most favorable physical advantages for the construction of a canal, but this is largely offset by the fact that, to quote the Commission, "the Nicaragua route is the more advantageous for all transisthmian commerce except that to or from the west coast of South America," and by the more healthful climate of Nicaragua. It is true that the Nicaragua project involves over a million dollars a year more for the expense of maintenance and operation; but this might be partially offset by the local development in the country which the canal will penetrate.

There are other features which are being urged in favor of the Panama route in the public press, which are not substantiated by the Commission's report. It is said, for example, that a Nicaragua canal will be in much more danger of injury by earthquakes than one at Panama. But the Commission, as the result of its careful investigation, could find no reason to suppose that the liability to destructive earthquakes was any greater on the one route than on the other, and on neither route did they consider the risk a matter of great importance.

Again, we hear it urged that the Panama route is the better because it can be made a sea level canal. But this matter also has been exhaustively investigated by the Commission. It has roughly estimated the cost of a sea-level canal as not less than $240,000,000 and the time required for construction at not less than 20 years. The Commission well says that the difficulties of all kinds with the sea-level project, and especially those of time and cost, make a canal with locks greatly to be preferred. As for De Lesseps' idea that a lock canal might be gradually transformed to a sea-level canal, we need hardly say to our readers that such a transformation would be absolutely out of the question. If a canal is to be built at Panama it must be built with locks, on the lines that the Commission has laid down.

Finally, there is one advantage on the side of Nicaragua which will yet, we believe, prove the determining factor—the fact that the United States can secure there a clear concession, encumbered by no past claims or obligations. This is far from being the case at Panama. It is hastily assumed by some that as the New Panama Canal Co. has now offered to sell all it has to the United States for $40,000,000, the way is now open for the immediate closing of the deal. Unfortunately for those who have sunk their money in the Panama canal, this is far from being the case. Before making any such purchase the United States would have to satisfy itself that the officers of the New Panama Canal Co. had the legal power to make such a transfer. It is still open to doubt whether the stockholders of the old Panama Canal Co. would not have the power at least to cloud the title given by such a transfer. Moreover, even if the Panama Canal Co. disappears, the question of a concession from Colombia is yet to be met, and the difficulty of making any satisfactory and binding agreement with the Colombian government at the present time is too obvious to require comment.

We said two weeks ago that President Hutin's resignation and the awakening of the Panama stockholders to their real situation comes too late for any proposition from them to secure favorable action by Congress; and we still believe this to be the case. Had the Panama Co. made a definite and reasonable offer to the Isthmian Canal Commission a year or more ago, that body would then have had time to thoroughly examine the legal status of the offer, and the State Department could have completed arrangements with Colombia. Will the Senate wait while these things are done, in the face of the public demand for immediate and definite canal legislation at the present session of Congress? The probabilities seem to us against it.

LETTERS TO THE EDITOR.

Another Reason for the Breaking of Suspension Rods on the Brooklyn Bridge.

Sir: The real cause of the breaking of suspension rods near the center of the Brooklyn Bridge was the intense stress these particular rods were subjected to by the abutting ends of the trusses at the expansion joint interfering with free vertical motion.

When one-half span is heavily loaded it descends, while the unloaded half span rises. As the ends of the half trusses move vertically, they interfere and cause intense stresses on the central rods.

The reason why the rods broke on the north roadway is because there was more vertical clearance in the south side expansion joint.

Yours truly,

Alexander Spotswood Dandridge.

New York, Dec 31, 1901.

A Peculiar Prismoidal Section.

Sir: Many engineers make the assertion that it is unnecessary to use the prismoidal formula in estimating earthwork. The following case occurred in this office, and may be of interest to some of your readers:

In excavating a railroad embankment for an undercrossing it was necessary to remove a prismoid whose upper surface was 120 × 12 ft., the lower surface 30 × 102 ft., and distance between them 30 ft.

An engineer of many years' experience, in estimating the quantity, used average end areas, and obtained as a result 2,500 cu. yds. I was requested to check the work, and by the prismoidal formula found it to contain 4,000 cu. yds., an increase of 60% over the first estimate.

The peculiarity of this section lies in the fact that the middle area is larger than either end area.

Yours truly, B. W. Hicks.

Ozark & Cherokee Central Ry.,
Fayetteville, Ark., Dec. 19, 1901.

Note on Rivet-Spacing Diagram.

Sir: In my contribution, "Another diagram for Rivet Spacing"(Eng. News, Dec. 5, 1901), you neglected to call attention to the fact that the formula for the minimum value of X holds good regardless of the value of d', which will vary should this happen to be a pin instead of a rivet. This fact was noted on the print I sent you, and is worthy of note, I think, as it is of constant use in determining rivet spacing near a pin point in tension members of a structure. Yours truly,

Arch. R. Eldridge, M. Am. Soc. C. E.

Chief Engineer's Office, C., B. & Q. Ry.,
Chicago, Ill., Dec. 9, 1901.

(The note should have been referred to in connection with the explanation of the diagram. It must be remembered, however, that while the upper, parobolic part of the curve will apply unchanged to the case mentioned, the lower part of the curve cannot be used, since it will give a spacing that provides a c. to c. distance of three diameters, i. e., 3 × ⅞-in. or 2⅝-in. in the curve shown, this being calculated for ⅞-in. rivets. With a pin as above mentioned by Mr. Eldridge, a c. to c. distance of three rivet-diameters would in general be insufficient.—Ed.)

Beware of Free "Useful Information."

Sir: Many dealers in engineering supplies endeavor to have their catalogues kept on file by publishing with them tables and information useful (more or less) to engineers. Some of these are carefully edited by men who appreciate the minus value of a mistake; but "there are others," in which the information is wrong, either absolutely or for the locality in which it is distributed.

As a recent example, we received yesterday the copyrighted (in 1896) catalogue of a steam pump company (we won't mention names), in the back of which are 3 pages of "useful information" and 21 tables. Table No. 13 shows the "weight and specific gravity of liquids," and tells us that the weight of one gallon of pure water is 10 lbs., of sulphuric acid, 18.4 lbs., and so on for about 30 liquids in the correct proportion to their specific gravities, also given. Of course, the engineer who remembers that (for rough measure) "a pint's a pound" and "eight pints make a gallon," will at once think this wrong—especially if he knows that this publisher, when he guarantees an efficiency for his pumps, does not calculate it in gallons containing 277.27 cubic inches; but there is nothing in the table to tell the unsuspicious American that the *British* gallon is referred to. The first column of the same table gives the weight of 1 cu. ft. of water as 62.425 lbs., while Table No. 10 tells us that its weight by "English standard" is 62.321 lbs.; so it could not have been an Englishman who "put *his* foot in it" when the international Table No. 13 was arranged. Moral: Unless you have time to discover and apply the "correction factors" do not use tables compiled by irresponsible parties.

Truly yours, L. F. Rondinella.

728 Stephen Girard Building, Philadelphia, Dec. 20, 1901.

Nicaragua or Panama.

Sir: Referring to your editorial comments in your issue of Dec. 26 on that portion of the report of the Isthmian Canal Commission covering the choice of route, your remarks convey the impression that this choice has been practically determined by the nature of President Hutin's correspondence on behalf of the Panama Co. and his refusal to answer unequivocally "What sum will you take?" If that is true, it is an exceedingly trivial matter thus to shape the great canal project.

Since there is a choice of two practicable routes, the United States is absolutely mistress of the situation. She should simply take the better of the two. This, though arbitrary, would involve no injustice. As you say, our country can afford to pay the full value to us of all property taken. If the Panama route is the preferable one, the Panama company should be told very plainly and finally that it will be given a certain sum only. Th's sum, if placed at the full value to us of the work done by the Panama company, will certainly be just and satisfactory to the latter under the existing conditions, for, otherwise, the Panama company must lose everything.

Now, the report of the commission shows that the present value of work done on the Panama Canal and the amount required to complete it are not greater than the necessary cost of the Nicaragua route. In favor, then, of the former are the much shorter length, lower level, and consequently greater efficiency and capacity. These are most valuable advantages, and, in fact, would justify the expenditure of a much greater sum for construction than would be advisable in the case of the Nicaragua route. We want and need the best canal. We could afford to pay an additional $50,000,000 to get it, but as the latter is not necessary let us have the best by all means.

The writer believes that the popular prejudice in favor of the Nicaragua route is unfounded, in physical facts at least; that Congress will commit an error in selecting the Nicaragua line, and that it is entirely within the power of Congress yet to acquire the Panama route property and franchises, and that the interests of the world as well as our own would be best served in so doing.

Respectfully, J. L. Campbell.

Room 915 Security Building.
St. Louis, Mo., Dec 28, 1901.

(We have commented on the above topic in our editorial columns.—Ed.)

Cost of Macadamized Roads at Wyoming, O., and Hudson, N. Y.

Sir: In your issue of Jan. 2 I notice an item on the cost of macadam roads at Wyoming, O. This article gives the average cost as 44 cts. per sq. yd. for 6 ins. of broken stone loose. This would correspond to a 4-in. roadway rolled. Perhaps it would be interesting to some of your readers who are interested in good roads to know our figures. We have constructed about 110,000 sq. yds. of macadam pavement in the last eight years. This work has been done both by contract and by day work. We are now constructing about 8,000 to 10,000 sq. yds. each year. The similarity between the cost per yard and the number of yards laid in Wyoming and Hudson leads me to write this letter. The only difference is that our specifications call for a 6-in. roadway after being rolled, and Wyoming about a 4-in. after being rolled. I notice by computing the amount of stone used at Wyoming and allowing 1½ tons to the cubic yard that Wyoming uses about ¼-ton per sq. yd. of finished pavement. Our figures give 3-10 of a ton to the sq. yd., indicating a heavier roadway. The average cost of our macadam, by contract, has been 44 cts. per sq. yd. During the past season I constructed 7,170 sq. yds. at an average cost of 42.3 cts. per sq. yd., slightly less than the contract cost. This was by day work. This price includes the laying of a 16-in. cobble or stone gutter on both sides of the street. Eight hours constituted a day's work on all day work, and ten hours on contract work; $12 a day was paid for the use of the roller on day work. One street comprising 4,228 sq. yds. was built by day work for 41.75 cts. per sq. yd. The cost of the different items on this street was as follows:

	Cents.
Hauling stone from the crusher to the street (1½ miles)	9.75
Cost of the stone crushed	12.75
Cobble gutters 24 ins. wide	5.07
Sprinkling	.28
Rolling	3.27
Spreading stone	2.58
Excavating and grading sub-grade	8.05
Total cost per sq. yd.	41.75

We have had very good success with our macadam roads. Our first streets laid have stood the traffic of eight years without being recrowned. The streets subject to the heaviest traffic will not need recrowning for a year or two. Yours respectfully,

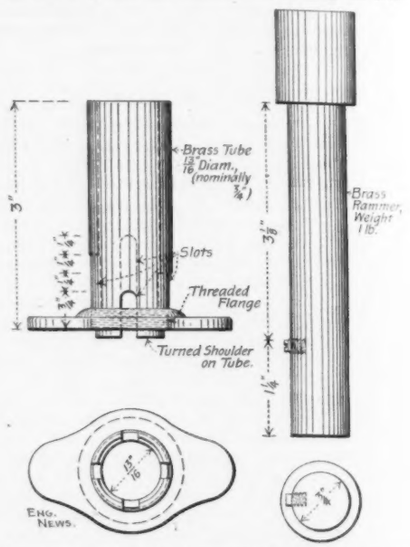
H. K. Bishop, C. E.,
Supt. of Public Works.

Hudson, N. Y., Jan. 2, 1902.

A Ramming Device for Compacting Cement in Briquette Molds.

Sir: In your issue of Sept. 12, 1901, there appeared the report of the Board of Engineers, U. S. A., upon the "Properties and Testing of Hydraulic Cement." This report was read with great interest by the writer, who is impressed with the thorough and comprehensive treatment of the subject. The method prescribed for compacting sand briquettes seems especially good, namely, to fill the molds in layers, each layer being compacted by dropping upon it a rammer of given weight and end area from a given height a given number of times. The method prescribed for operating the rammer is to lift it with the fingers and let it fall apparently without the aid of mechanical means for controlling the height to which it is raised and the squareness of the blow. It would seem to the writer that to obtain good results mechanical aid is necessary, and he has endeavored to supply it in designing and constructing the apparatus shown in the accompanying sketch.

This apparatus consists of a rammer and a casing in which it works. The rammer has a round section ¾-in. in diameter at the striking end, weighs 1 lb. and is fitted with a screw plug to act as a stop to regulate the height of the blow. The casing consists of a brass tube having four longitudinal slots cut upward from the bottom at equidistant points on the circumference. A flange screwed on to the base of the tube supports the apparatus on the mold. The bottom end of this tube, turned down as thin as is consistent with the strength required, is carried a little below the flange in order to keep the apparatus in the mold and to prevent the rammer from striking the side



Ramming Device for Compacting Cement in Briquette Molds.

The slots are of such lengths as to catch the stop of the rammer when the end of the same is ½-in. below, ¼-in. below, level with, and ¼-in. above the top of the mold, the stop being introduced in any slot by pushing the rammer down in the tube as far as it will go and turning it until the stop comes in line with the slot. The arrangement allows of ½-in. drop on each ¼-in. layer. The rammer is lifted and dropped 30 times on each layer, the apparatus being moved continuously so as to distribute the blows evenly.

The final layer is placed and compacted by the aid of a second mold placed on top of the one in which the briquette is being made. The apparatus is placed on this mold, and, using slot which permits the rammer to rise to a point ½-in. below the top, a ½-in. blow is struck on the final layer in the mold below.

This apparatus is being used in the Cement Testing Laboratory of the N. Y. C. & H. R. R. R. with good success. It is found that it takes a good deal of time to make a briquette in four layers so that experiments in the line of using two layers with a greater drop are in progress.

As to the need of such an apparatus, the writer would state that the old method in use in this laboratory and which was hitherto considered the best practical method has been to place mortar in two layers, the first of which half filled the mold, when compacted by striking a certain number of blows with a light hammer of ¾-in. round head. The second layer was heaped over the top of the mold and a wooden block placed on top was struck a certain number of light blows as before. This method did pretty well, but depended largely on how strong the operator felt at time of making the briquette, which in consequence varied considerably in compactness and in resultant strength. Very truly yours,

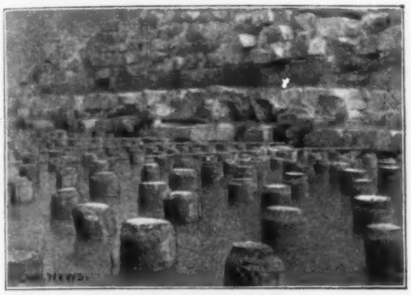
Allan W. Carpenter,
Assistant Engineer, N. Y. C. & H. R. R. R.
Grand Central Station, New York City, Dec. 8, 1901.

The Decay of Piles Embedded in Concrete and Cut Off Above Low Water Mark.

Sir: The accompanying photograph shows a phenomenon of interest to me, and presumably to other members of the profession as well. The reader will note the appearance of decay around the pile heads. The circumstances are as follows:

Some 13 years ago the Boston Electric Light Co., or a predecessor, erected a power plant on what was then the water front at the foot of Summer St., Boston. This plant consisted of a dynamo and engine house, a boiler house adjacent, a chimney, and a coal wharf, with bins and hoists. The recent construction of the South Station by the various railway companies entering the city from the south necessitated several street changes, which cut off this plant from its tidewater coal supply, and probably partly on this account the machinery was moved to a more favorable location. Within about a year past the

site has changed hands and new buildings are being erected. In excavating for the new foundations these piles were uncovered. There seems to be ample evidence that they were well driven into the bed of clay of varying hardness which underlies this section of Boston, and were cut off well below any local injury to the heads caused by driving. They were braced by about a foot of concrete around their heads. On this concrete was laid a triple layer of heavy granite blocks, each 18 to 20 ins. in thickness, which had become cemented together almost as hard as solid rock. This granite bed underlaid the whole area of the engine and dynamo room and carried the brick walls of the building and brick piers about 8 ft. high for each machine. Notwithstanding all these precautions, the vibration of the engines and dynamos was very noticeable in adjacent buildings whenever the station was under heavy load, as, for instance, about 4:30 p. m. on winter days. This location being so near the harbor, the ground water level fluctuates with each tide, rising some 2 ft. above and falling some 3 ft. below the tops of the piles. The surrounding buildings are occupied partly by printers and partly by light manufacturers. The illustration shows a near view of the pile heads. On the nearest pile may be seen a 4-in. pocket rule, which gives some idea of the dimensions and the extent of the decay.



View Showing Decay of Heads of Piles Which Had Been Embedded in Concrete.

If this illustration shows a fair sample of what takes place in a mid-tide foundation in about 13 years, it would be interesting to know how it would look in 50 years. It would also seem important to know why these piles decayed. My hypothesis is that air or gas followed the receding tide through the porous filling under the concrete, and rising around the piles, displaced the water, allowing a partial drying out and consequent decay. It will readily be seen that this might take place even in case the foundation were below low tide if the filling or ground contained enough gas; and dock mud is seldom free from it.

As others may be interested and may be able to contribute something to the general knowledge of piles under concrete, I send the above for publication.

Yours truly, H. K. Higgins,
Asst. Engr. N. Y., N. H. & H. R. R.

470 South Station, Boston, Mass., Dec. 21, 1901.

(The only safe rule is to cut off piles below low-water level. No foundation can be called permanent in which wood is used where it may be exposed to the air. Nothing happened in the case shown in the photograph that any competent engineer would not have expected and predicted. The surprising thing to us is that in the enlightened city of Boston an important structure should be built on a heavy concrete and granite foundation evidently intended to be as permanent as the hills and yet 3 ft. of wooden piling at the bottom of the whole was placed where it was certain to decay. It is clearly a fortunate thing for all parties concerned that the power station was moved.—Ed.)

Contract Tests of Mechanical Stokers at the General Electric Works, Schenectady, N. Y.

Sir: There appeared in the Nov. 7 issue of Engineering News an article under the above heading which consisted of extracts from the report of a series of tests made by Prof. J. E. Denton and Mr. George H. Barrus, representing the American Stoker Co., and Messrs. Dean & Main, of Boston, representing the General Electric Co. Owing to the manner in which these extracts were presented by Engineering News, and the absence of reference to several important facts connected with the tests, the article in question does not present the case in a manner equally fair to all parties concerned.

As stated in the report, these tests were conducted for the purpose of determining whether the American Stoker Co. had fulfilled certain guarantees in their contract with the General Electric Co. On account of a clause in the contract, in which the American Stoker Co. guaranteed to "generate a net horse-power of steam as economically as the Roney stoker," the engineers decided to test one of the Babcock & Wilcox boilers equipped with the Roney stoker, which had been installed in 1898 and which had been in operation continuously, day and night, for over 2½ years.

The position was taken by the engineers that as the tests were to be conducted for the purpose of determining whether the American Stoker Co. had fulfilled its guarantees, the manufacturers of the Roney stoker had no interest or part in the matter, their stokers having been accepted and paid for several years before. Under this ruling the Roney stoker was brought into a competitive test in which the makers were not permitted to direct its operation and were not represented except by an erecting superintendent, who was allowed to witness the final test, but not to give directions. One of the results of this ruling is shown in that part of the report regarding "labor required for operating the stokers," where the report states that "in the Roney stokers, according to the practice of the fireman during the test, there was continual attention and exertion required, etc." This method of handling the stoker was contrary to the "instructions for operation" which are furnished with the Roney stoker, and was the result of inexperience, the fireman having never fired on a test before. If the manufacturers had been permitted to direct the operation of the stoker, this unnecessary labor would have been avoided.

Under the paragraph, "Comparative Economy of the American Stoker and Roney Stoker," various costs of operation are summed up as follows:

	American stokers		
Costs per HP. per year.	Repairs according to 3 mos. record on log book.	Repairs guaranteed in contract.	Roney stokers.
Cost of coal	$34.95	$34.95	$35.76
Cost of repairs	0.77	0.12	0.48
Wages of fireman and helpers.	1.44	1.44	1.44
Interest and depreciation	0.38	0.38	0.38
Total cost per HP. per year.	$37.54	$36.89	$38.06

In the above table it seems rather illogical from an engineering standpoint to credit the American stoker repair account with the excess over 5%, which the makers, under their contract, assumed for a period of two years. The question of financial liability will not be apt to have much effect on the actual deterioration of the stoker.

As the report assumes that this table demonstrates that "the cost of steam is less with the American stoker than with the Roney stoker," it is but fair to call attention to some facts not brought out in the report which bear on the other side of the question.

The principal item in the table is "cost of coal" per horse-power per year. This is obtained by dividing the amount of water required per horse-power for 300 days of 24 hours each by the net evaporation from and at 212° F. per pound of coal. This method of determining the efficiency of the stoker by means of the boiler performance is manifestly an improper one. The condition of the boiler or the arrangement of the heating surface may be such that its inability to properly absorb heat will neutralize the most perfect combustion obtainable in the furnace. If in these tests the boiler evaporation was to be assumed as the standard of stoker efficiency, the boilers should have been of the same type and the heating surface and settings in the same condition. The Babcock & Wilcox boiler and Roney stoker had been in use over 2½ years, and had never had the fire sides of the tubes cleaned except by blowing from the outside, and the setting was more or less cracked, while the Stirling boiler with American stoker was new and recently put in service. A contract test was made some six months after this Babcock & Wilcox boiler and Roney stoker were put in service, at which an evaporation of 12.66 lbs. of water from and at 212° per pound of combustible was obtained, exceeding any of the results obtained by either stoker in the series of tests under consideration. Substituting this evaporation for that obtained on June 18, 1901, when the boiler was old and dirty, would make the cost of coal required per horse-power per hour $34.08 instead of $35.76, as given in the above table. If the table be corrected to make the "cost of coal" correspond with what it was when the boiler was new and clean, the "cost per horse-power per year" of operating the Roney stoker would be $36.40 per horse-power instead of $38.06, as given in the table, and less than either of the amounts given as "cost of operating" the American stoker.

The "cost of repairs," namely, 48 cts. per horse-power per year, is excessive, on account of the fact that the draft of the Babcock & Wilcox boiler was only 0.15 to 0.16-in. in the furnace, due to insufficient chimney. These boilers and stokers are operated with natural draft, and the poor draft caused high temperature in the ashpit and excessive repairs. The records of many large plants show that the cost of grate bar repairs for the Roney stoker, with proper draft conditions, should not exceed 12 cts. per HP. per year.

A stoker does not evaporate water. Its function is to burn coal, and the measure of its efficiency is the ability to produce good combustion as shown by the analysis of the flue gases and not the amount of water evaporated per pound of coal by the boiler to which it is attached. The record of the analyses of flue gases from the Roney stoker shows an average of CO_2, 14.2; O, 4.67; CO, 0.29.

This is as good combustion as has ever been obtained, and the fact that the evaporation from and at 212° F. per pound of coal by the Babcock & Wilcox boiler and Roney stoker was 11.169 instead of 11.7, as obtained when the same boiler was new and clean, simply shows that the difference is due to the fact that the boiler was not in condition to absorb the heat generated by the stoker. The heat balance given in the report shows that the Stirling boiler absorbed 77.1% and the Babcock & Wilcox boiler 72% of the heat of combustion. Or, in other words, judged by the ability of the boilers to absorb heat, the Roney stoker was handicapped by a difference in boiler efficiency of 7%. This is more than double the amount given in the table quoted above as the difference between the two stokers in "cost of steam per HP. per year," and completely reverses the statement in the report that "in point of relative economy the guarantee of the American Stoker Co. is fulfilled." It is most important in comparing these tests to remember that the boiler to which the Roney stoker was attached was old and dirty, and that the combustion was of the best. In comparing the economy of the two stokers, the operation of the boilers should have been eliminated and the efficiency of the stokers determined by their ability to produce good combustion. To be consistent, the experts should have considered the results of the tests from this standpoint. Had they done so their conclusions would obviously have been different.

William R. Roney.

26 Cortlandt St., New York, Dec. 18, 1901.

Methods of Specifying the Relative Proportions of Sand and Cement for Mortar and Concrete.

Sir: Referring to the article on "Estimating the Cost of Concrete," by H. P. Gillette, in your issue of Dec. 5, I enclose the accompanying diagram showing the volume of mortar formed with various mixtures of sand and cement. The data were compiled from actual measurements, made by different parties, as noted on the diagram. The specifications for the regulating works on the Chicago Drainage Canal called for a concrete proportioned so that the volume of broken stone used was in proportion to the volume of mortar formed. The canal experiments are indicated by a part of the points near the 1 to 1 and 2 to 1

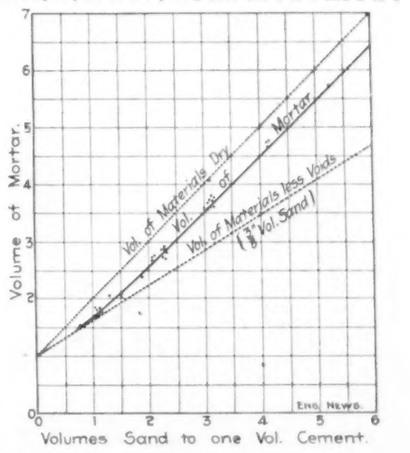


Diagram Showing the Amount of Mortar Formed with Various Mixtures of Sand and Cement.

mixtures. The other points along the curve are from the Report of Engineers U. S. A. 1895. In the canal experiments the points are platted for cement packed in the condition of 3.6 cu. ft. per barrel. The diagram confirms Mr. Gillette's theory "that all voids in sand are not filled by cement."

A word in regard to specifications for a definite amount of cement in mortar. In practice the various methods describing the proportions for mortar are as follows: (1) Volume of sand to 1 volume of cement; (2) weight of sand to weight of cement; (3) volume of sand to weight of cement; (4) volumes of sand to 1 volume of cement of a specified weight.

None of these methods insure a fixed or definite amount of cement. This is an important point when it is considered that a variation of 10% in the amount of cement will change a nominal 1 to 8 concrete to a 1 to 9 concrete. Take method (1). If the unit of volume is cement in the original package, the amount of cement in a cubic foot will depend on the packing at the mill. Measurement by volume of cement in this condition is subject to as little variation as with any of the four methods. If, however, the cement is measured loose, as is often done, the amount is very indefinite. A glance at the varying figures for cement per barrel, loose or packed, that are given in Mr. Gillette's article, clearly shows the objections to method (1). In methods (2) and (3) the amount of cement is specified as so many pounds. This method is free from any variation due to compactness and would be perfect if all brands of cement had the same specific gravity. Method (4) is an attempt to regulate the compactness of

I would, however, recommend that for all work of any magnitude experiments be made with the actual materials to be used to determine values for "m," "b" and "v;" or better yet to determine "N" direct if this is practicable on a sufficiently large scale.

I have recently had some experience with concrete made of cement, gravel and crushed stone, the gravel containing the sand necessary for mortar. The proportion of sand in the gravel as delivered ranges from 30% to 80%. It is necessary to make a screening test of every shipment of gravel received in order to determine the relative amounts of gravel and stone required to yield the desired equivalent proportions of cement, sand and aggregates.

The relative proportions of sand and pebbles in different parts of the same pile will vary considerably owing to the tendency for the larger pebbles to roll down to the foot of the slope, so it is impossible to make as uniform a mixture as with cement, sand and crushed stone. However, the mixture of smooth pebbles with the crushed stone is easy to work and makes a very compact concrete, giving a good smooth face by working a spade up and down just inside the form to bring the mortar out to the front.

H. P. Boardman, Asst. Engr., of Bridges and Buildings, C., M. & St. P. Ry.

Chicago, Ill., Dec 28, 1901.

A General Review of the Discussion on Proposed Changes in the New Croton Dam.

Sir: Mr. A. Fteley, in his reply to the Board of Experts on changes in design of the New Croton Dam, in Engineering News of Dec. 12, shows very conclusively why such changes as have been recommended are not necessary.

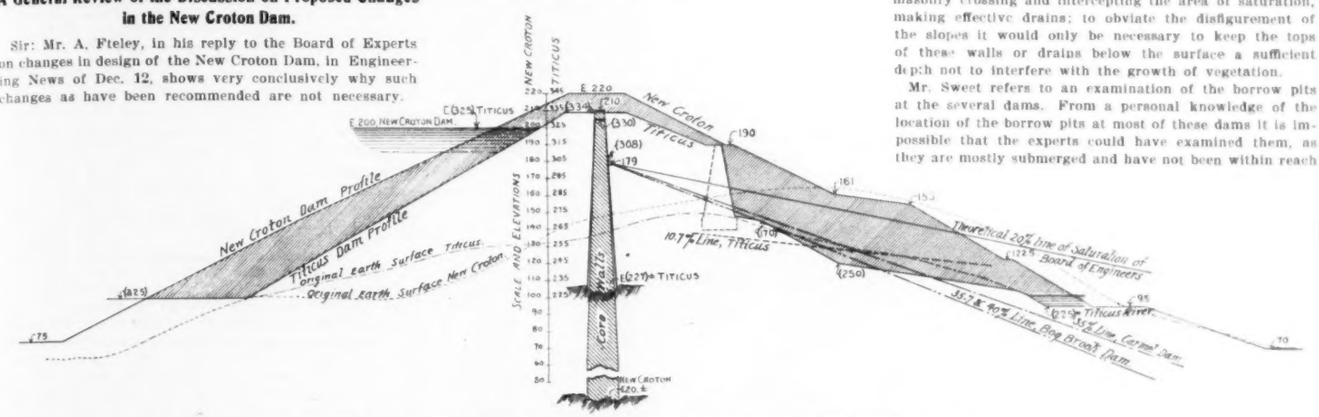

New Croton Dam Profile
Titicus Dam Profile
Original Earth Surface Titicus
Original Earth Surface New Croton
SCALE AND ELEVATIONS
Core Walls
10.7% Line, Titicus
Theoretical 20% line of saturation of Board of Engineers
35.7 & 80% Line, Bog Brook Dam
35% Line, Carmel Dam
Titicus River

FIG. 1. COMPARATIVE PROFILES OF NEW CROTON AND TITICUS DAMS.

(These are developed profiles, on "lines of least resistance to the passage of water." The elevations in parentheses relate to the Titicus Dam.)

Mr. Elnathan Sweet, in your issue of December 26, however, takes exception, among other things, to Mr. Fteley's remarks on the comparative profiles of the New Croton and Titicus dams, as developed on "lines of least resistance," in which Mr. Fteley claims that a juxtaposition of these profiles will show a result favorable to the New Croton.

Evidently on this profile (Fig. 2 Experts' Report) the experts have mainly based their contention that the earth dam should not be constructed, owing to its insufficiency in outside embankment to contain the area of saturation that they claim will follow the filtration of water through the inner bank and a leaky or pervious core wall.

Mr. Sweet objects to a comparison of the New Croton and Titicus profiles, overlooking the fact that in his memorandum filed with the experts' report he has especially pointed to the Titicus as "the boldest example of this type" of dams, and then follows with a comparison of the two profiles.

He claims the New Croton embankment is twice as high as that of Titicus. He has given a definition of the effective height of a dam embankment "as the difference in elevation of the highest water in the reservoir and the elevation of the intersection of the outer slope with the natural or restored surface of ground below the dam."

It is evident, therefore, that the New Croton embankment is only about 30 ft. higher than that of Titicus, instead of twice as high. It is true that the New Croton core wall has nearly twice the height of that at Titicus, but in the very nature of this case, where both walls are carried to the underlying rock, this difference in height is more apparent than real; more properly speaking the difference is in depth below the base of the dams.

It will be seen by Fig. 1 that there is only 30 ft. difference in embankment height: New Croton has an interior slope of 2 to 1, Titicus of 1¾ to 1; New Croton on its outer slope has a variable profile much increased in value by wide bermes carrying it far beyond the profile of Titicus, which also has a variable profile abruptly broken by a wing wall, which was not intended as, and is not a barrier to filtration.

This wall has only a shallow foundation, and as will be seen by reference to Fig. 1, the slope of saturation for the Titicus, as established by the experts, will pass under this wall and out through the restored surface beyond, which in accordance with their theory renders the bank unsafe.

The excess of New Croton profile over Titicus, as may be plainly seen, is due to its greater width at water line, the New Croton having a width of 115 ft., the Titicus of only 72 ft. The measurements are on the developed "lines of least resistance." (The actual widths on normal sections are 110 and 74 ft.; see Fig. 2.)

This excess in the New Croton is divided between the outer and inner banks, and it shows largely in favor of the New Croton Dam, as was stated by Mr. Fteley.

Fig. 2 shows comparative sections of the two dams at right angles or normal to the slopes, taken, in each case, near the dividing lines between the earth and masonry dams. It represents truly the actual differences in sections, which differences, as in the case of the developed section, are unquestionably in favor of the New Croton embankment. It shows a dam which, by Mr. Sweet's definition of effective height, is only about 40 ft. high. This is just south of the heavy wing wall.

The developed profiles on "lines of least resistance" (Fig. 1) have been used in making comparisons only for the reason that they are a creation of the experts and it is the only method by which the New Croton embankment could be made to appear higher than any of the dams now in successful service in the Croton valley.

Assuming, however, that the outer refill is to be treated as a part of the bank proper: Then, on Fig 1, are produced the several slope lines of saturation as determined by the experts for the other dams.

The theoretical 20% line of the experts would indicate a danger point on the developed profiles, while the lines of Bog Brook and Carmel dams indicate absolute safety, passing far below the 10 ft. limit of the slope of the bank, a limit of safety fixed by the experts.

On the other hand, all the lines indicate unsafety in the Titicus profile. These lines are plotted with the experts' assumption of a loss of head of 17% of the depth of water in the reservoirs; with the Titicus depth of water they would be 4 ft. higher, beginning at a point marked E (308).

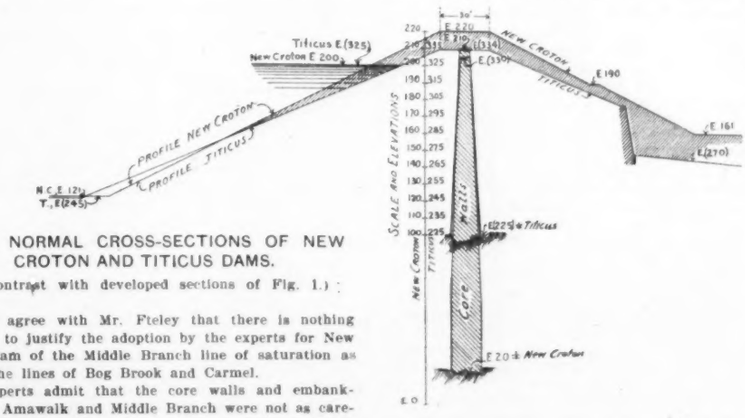


FIG. 2. NORMAL CROSS-SECTIONS OF NEW CROTON AND TITICUS DAMS.

(In contrast with developed sections of Fig. 1.)

I fully agree with Mr. Fteley that there is nothing whatever to justify the adoption by the experts for New Croton Dam of the Middle Branch line of saturation as against the lines of Bog Brook and Carmel.

The experts admit that the core walls and embankments of Amawalk and Middle Branch were not as carefully constructed as those of the other dams, yet they arbitrarily select the Middle Branch line of saturation to apply to the New Croton Dam.

I contend, and with a full knowledge of the work, that the wall at the New Croton is being built in every respect fully equal to the walls at Carmel, Bog Brook and Titicus, and having about the same thickness as the latter it should be equally impervious to filtration.

Regarding Mr. Sweet's assertion that this core wall shows an ineffective resistance to water under a moderate head, I have the following statement from the Resident Engineer in charge of the New Croton Dam:

In referring to leaks through the core wall Mr. Sweet evidently has in mind the seepage which showed itself at or near the bottom of the well which was built in the wall near its junction with the main dam. At this elevation the wall proper is about 12 ft. thick, the well being 4 ft. square, leaving 4 ft. for the thickness of its side walls. It was found upon investigation that in the well there was nearly 6 ft. of water which had entered through a drain pipe leading from a point near its bottom through the down-stream side of the wall into the fill placed against it. When the well was emptied of the water the seepage gradually disappeared.

At Middle Branch the test wells indicate a greater loss of head inside of the core wall than outside; also at Bog Brook there appears no loss of head in passing the core wall. If these conditions are correctly shown it is very peculiar. It makes it appear that the experiments may be at fault, or there are other causes of saturation than leaky core walls. I am of the opinion that the water in these banks is largely due to natural causes, and that the past season, covering a period of heavy and frequent rains, has had its influence on the conditions found. Undoubtedly tests made in the natural hill slopes will show practically the same conditions, varying as does the material and inclination of the slopes. The uniformity of the slopes of saturation in the artificial banks is due to the uniformity of material and construction.

Admitting again for argument that saturation may reach the surface of the bank slopes, it is a simple and inexpensive matter to build, for instance, dry walls of masonry crossing and intercepting the area of saturation, making effective drains; to obviate the disfigurement of the slopes it would only be necessary to keep the tops of these walls or drains below the surface a sufficient depth not to interfere with the growth of vegetation.

Mr. Sweet refers to an examination of the borrow pits at the several dams. From a personal knowledge of the location of the borrow pits at most of these dams it is impossible that the experts could have examined them, as they are mostly submerged and have not been within reach of observation for a considerable period, and by far the best material came from within the basins. Those now visible do not give even a fair measure of the general character of the bank material.

The writer had direct personal charge of the construction of the Carmel and Titicus dams for a period of five years, residing in their immediate vicinity, with full opportunity for daily observations of all details of construction. He also had, and availed himself of, opportunities to observe the work of construction on the Bog Brook, Amawalk and New Croton dams, and is naturally more conversant with the real conditions than could possibly be had by a few visits to these works already completed (excepting New Croton) and in service and beyond the reach of any but a very superficial examination of the banks.

A lack of precedent has been strongly urged as a reason for not constructing a dam of earth somewhat over 100 ft. in height. If we wait for precedent surely no advance will be made in any form of construction. There is absolutely no reason why we should stop at 70, 100 or 200 ft. in height for an earth dam; it is merely a question of necessity or expediency. The profession of engineering and architecture would long ago have been at a stand-

the unit volume of cement. It is a step in the right direction, but is imperfect in that it allows variations in compactness with cements of different specific gravities. For example, if a cement is specified to weigh 70 lbs. per cu. ft., a light cement thoroughly packed will conform to the specifications, as will a heavy cement loosely packed; but the volume of mortar obtained from the heavy cement will be less than that made from the light cement.

What is wanted in the work is a definite volume of cement in its most compact form or as it exists in the voids of the sand in mortar. What the weight of this volume of cement is (in other words what its specific gravity is), we do not care, for the strength of the mortar depends on the volume of solid cement used and not on the weight or specific gravity. If we consider the condition of thoroughly compacted cement to be equivalent to neat cement mortar of the same consistency as mortar for testing briquettes, then we have a basis for a definite amount of cement that is independent of personal judgment or variations due to method of handling.

The writer would suggest the following specification for proportioning mortar:

The mortar shall consist of one part cement to —— cu. ft. of loose sand. Said part of cement shall be the amount sufficient to make 1 cu. ft. of neat cement mortar of the same consistency as the mortar used for making the specified tests of tensile strength.

L. K. Sherman.

466 La Salle St., Chicago, Ill., Dec. 10, 1901.

Concrete Materials and Proportions.

Sir: I was much interested in the article on "Estimating the Cost of Concrete," by H. P. Gillette, in Engineering News of Dec. 5. I have some records of my own bearing on the subject which may be of interest to your readers.

In 1900, while with the Chicago & Alton Ry., I assisted in drawing up concrete specifications for that road, and a few quotations from those specifications are given below:

Concrete will, in general, consist of a matrix of cement mortar and an aggregate of broken stone or gravel, or a combination of broken stone and gravel.

MATERIALS.—The cement used shall conform to the C. & A. Ry. Co. specifications.

The sand shall be practically free from clay, loam, sticks, leaves and other foreign substances, but may contain occasional pieces of small gravel. The sand shall be so coarse that not more than 40% of it will pass through a No. 50 sieve, 50 meshes per lin. in.

The broken stone shall be of sound limestone, which shall be composed of angular fragments, no piece of which shall exceed 2 ins. in greatest dimension. It shall be clean and free from dust, dirt and other foreign matter, but may consist in part of fine screenings and medium-sized pieces, the intention being to take all of the product of the crusher except fine dust.

The gravel shall be clean and free from dust, dirt and other foreign substances, and shall contain no stones over 2 ins. in diameter. When gravel containing considerable quantities of sand is used several trials must be made to determine the proportion of sand it contains that will pass through a No. 4 screen and a corresponding deduction shall be made from the sand used in making the mortar, so that the final proportions of the cement and sand in the complete mixture shall be as specified for mortar. Only the gravel remaining after screening out the sand shall be treated as aggregate during the experimenting for voids referred to later.

PROPORTIONS.—When the cement is packed in barrels the barrel shall be the basis of measure for the sand and also for the aggregate. But when the cement is packed in sacks, 3½ cu. ft. shall be considered a barrel of sand or aggregate.

When the aggregate is composed of both broken stone and gravel they shall be mixed in such proportions (to be determined by experiment) as will give the least per cent. of voids when mixed.

In general, the amount of mortar mixed with the aggregate shall be at least 10% more than the voids in the loose aggregate, which the engineer shall determine by experiment.

In experimenting for voids, where possible, the method of weights shall be used assuming the weight of dry gravel and broken stone to be 165 lbs. per solid cu. ft., and excluding from the aggregate the sand which occurs in the gravel.

The amount of mortar made by given amounts of cement and sand shall also be determined by experiment, but in the absence of such experiments for special cases the following may be used as an approximation, the units of cement being a barrel of 380 lbs. net for Portland and 265 lbs. net for natural cement:

	Cement— Portland.			Natural.		
Proportions	1 to 1	1 to 2	1 to 3	1 to 1	1 to 1.5	1 to 2
Cement, bbl.. ...	1	1	1	1	1	1
Sand, cu. ft.....	3.5	7.0	10.5	3.5	5.25	7.0
Resulting mortar, cu. ft...	6.0	8.0	10.7	5.7	6.9	7.8

In all cases the proportions shall be reduced to such a basis that they can be stated as follows in terms of the actual materials as delivered on the ground for use:

One bbl. cement to —— cu. ft. sand to —— cu. ft. gravel to —— cu. ft. broken stone.

The proportions so stated, together with descriptions of all experiments leading thereto, shall be submitted to the chief engineer for approval, and after approval shall be followed in measuring the materials until there is reason to change them through change in condition of materials received. The necessity for such change shall be determined by the engineer and approved by the chief engineer.

In writing these specifications the intention was to do away as far as possible with the uncertainty of the ordinary way of stating proportions as 1 cement, 3 sand and 7 stone. As pointed out by Mr. Gillette in the article referred to, the volume of cement varies so greatly with the conditions that specifications should leave no doubt as to the units of measure to be used.

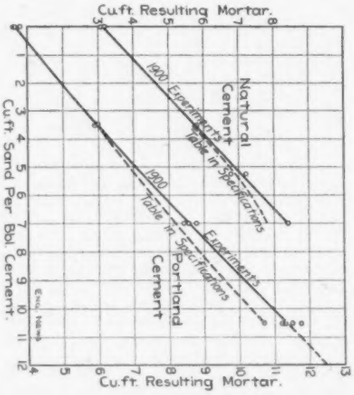
By specifying a barrel as the unit of measure of the cement and defining a barrel as 380 lbs. net for Portland and 265 lbs. net for natural cement would seem to fix the cement all right providing different brands of cement of the same class yield the same amount of mortar for a given weight of cement, other conditions being the same. This last is substantially true as far as the writer has been able to determine. In practice the barrel is a good unit of measure for cement, it being usually delivered in barrels or in ¼-barrel sacks. Short-weight sacks or barrels will have to be guarded against in measuring out cement.

As to the variation in volume of sand, a startling statement is made on page 84 of the 1899 edition of Baker's "Masonry Construction," to the effect that the addition of 2% by weight of water to dry sand increases the volume of the sand by 20%. This statement I was unable to believe offhand even after looking up one of the references given, so I experimented a little for myself. I found an increase in volume by thoroughly mixing the given amounts of water and sand, about as follows:

Water added, %.....	2	4	6	8	10
Increase in vol. of sand, %.	17.6	22	19.5	16.6	15.6

I, however, discovered that a very moderate amount of shaking would reduce this increase in volume about one-third to one-half. This still leaves a very noticeable increase in volume.

The table of resulting volume of mortar from certain amounts of cement and sand, as given in the above specifications was based on some experiments on a small scale made in the cement-testing laboratory of the Sanitary District of Chicago in 1895 and 1896, but the volume of resulting mortar was cut down somewhat to be on the safe side. This was on the safe side, since an underestimate of the amount of mortar, when used in determining the proportions in the manner directed would give proportions which would yield a concrete richer in mortar than was specified. Not being entirely satisfied with the table, I made other experiments on volume of mortar, but did not get the results soon enough to embody them in the specifications. These results are shown graphically (1900 experiments) in the accompanying diagram. The curves, representing the values given in the table of the specifications, are also shown.



Results of Experiments to Determine Volume of Mortar Produced by Varying Proportions of Sand and Cement.

Nearly always the sand used in making concrete in large quantities is somewhat damp; so in making these 1900 experiments for volume of mortar I used damp sand, containing 1½ to 3% by weight of moisture and aimed to compact it by shaking to just about the extent it would be compacted by handling in larger quantities as in measuring out for concrete.

Experiments were made by concrete inspectors in the field which checked these laboratory experiments very closely, though in one case an inspector was unable to get as low as 12 cu. ft. of mortar from 1 bbl. cement and 10.5 cu. ft. sand until, after repeated trials, he finally used water enough to make a wet mortar, when the resulting volume was about 11.4 cu. ft.

The amount of mortar obtained depends on various factors: the kind of sand (fine or coarse); the amount of moisture it contains, and how much it is compacted when measured; the proportion of voids; the care used in mixing cement and sand; the amount of water used, and the tamping the mortar finally receives. The tamping required to give the most dense mortar varies inversely as the amount of water used.

These seem to me the most common causes of variation in the mortar, and though they are numerous, if a certain standard of practice be adopted the results should be fairly uniform and I think the curves given in the figure represent a fair average of what should be obtained in practice.

As before stated, I do not think the brand of cement greatly affects the volume of resulting mortar so long as weight is the unit of its measure, and I think the weights given, namely, 380 lbs. net for Portland and 265 lbs. net for natural cements, are the most commonly accepted weights per barrel.

The experiments referred to as made in 1895 and 1896 were with Empire, Wayland and Giant Portland and Louisville natural cements, while those made in 1900 were with Lehigh, Wolverine, Atlas and Vulcanite Portland cements and Louisville and Milwaukee natural cements.

As to the causes affecting the volume and other characteristics of concrete, practically the same remarks as made for mortar may be applied. Especially is this true in regard to the amount of water used, and I believe that in general enough water should be used to insure quaking of the concrete under comparatively slight tamping. This is enough, but a little more water does less harm than a little less water. As to Mr. Gillette's formula for the number of barrels of cement required to make 1 cu. yd. of concrete, viz.:

$$N = \frac{27}{p + 1\,[\,(1 - 0.8\ v)\ \Psi + (1 - 0.8\ V)\ Y\,]}.$$

I do not fully agree with him in the value of some of the constants. He says: "The number of cubic feet of cement paste per barrel usually equals the volume of cement packed in the barrel."

I have seen but little cement used from barrels recently, but in 1900 I measured several Lehigh cement barrels and found their capacity to be about 3.2 cu. ft., while my result for neat cement mortar from Lehigh cement was 3.65 cu. ft. per bbl. My experience with voids in sand and crushed stone is usually about as follows: sand (loose), 34% to 40%; crusher run stone (loose), 39% to 42%; screened stone (loose), 48% to 52%.

Where the approximate weight per solid cu. ft. is known for the particular stone, gravel or sand being used, I prefer the method of weights for determining voids. Pure calcite weighs about 170 lbs. per cu. ft., and pure quartz about 165 lbs. Tables of weights of limestones give weights ranging from 150 lbs. to 175 lbs. per cu. ft., and even beyond these limits in rare cases. I think a fair average for most of the sand, gravel and limestone used in making concrete, at least in the Mississippi valley, is 165 lbs. per solid cu. ft. dry. I have tested four or five kinds of stone and sand used for concrete and they ranged from 159 lbs. to 166 lbs. per cu. ft.

A variation of 5 lbs. per solid cu. ft. either way from 165 lbs. will change the proportion of voids calculated by weight to the extent of 1½% to 2%, which is not a serious matter in determining concrete proportions.

Again referring to Mr. Gillette's formula, let us compare a part of it with the results of experiments as represented by the 1900 curve for Portland cement shown on the diagram.

Let p = 3.65 = cu. ft. neat cement mortar per bbl. cement.
" c = lx = cu. ft. sand mixed with 1 bbl. cement.
" s = ly = cu. ft. of aggregate per bbl. cement.
" m = cu. ft. mortar obtained per bbl. cement with different proportions of sand.
" a = proportion of voids in sand that are filled with cement mortar, assumed by Mr. Gillette = 0.8
" d = (1 — av).
Assume v = .37 = proportion of voids in sand.
Then from Mr. Gillette's formula, $m = p + 1 \times (1 - 0.8\ v)$
Making the substitutions as above we have

$$m = 3.65 + c\ d,$$

$$d = \frac{m - 3.65}{c} \quad \text{and} \quad a = \frac{1 - d}{v}.$$

Taking c and m from the curve and deducing d and a we have the following table:

c	m	d	a
2	4.95	.65	.95
5	7.05	.68	.87
8	9.35	.71	.78
11	11.75	.74	.71

Thus we see that according to these experiments a, the proportion of voids in the sand which are filled with cement, varies considerably, being largest for rich mixtures, as would seem natural.

However, the same process applied to the corresponding curve for natural cement gives nearly a constant value for "a," ranging only from .68 to .70.

Now, considering Mr. Gillette's complete formula and making substitutions, we have:

$$N = \frac{27}{m + s\ (1 - b\ v)} = \text{Number of barrels cement required per cubic yard of concrete.}$$

Where m and s are as given above, m to be taken from the curves.
v = the proportion of voids in the aggregate.
b = the proportion of voids in the aggregate that are filled with mortar.

By applying this formula to some rather incomplete data which I have, I find that for Portland cement concrete "b" varies from .82 to .95, being above .90 in 3 out of 4 cases, and for natural cement concrete for which I have less data "b" is between .70 and .75.

Then using Mr. Gillette's style of formula and eliminating all the theory we can, substituting therefore according to the above experiments and data from practice, we have for Portland cement concrete

$$N = \frac{27}{m + s\ (1 - .9\ v)}.$$

and for natural cement concrete

$$N = \frac{27}{m + s\ (1 - .75\ v)}.$$

The canal is 22 ft. deep, 98 ft. wide at the bottom and 18 miles long, from the Gulf to Konigsberg. The course of the River Pregel, on which the city stands, has also been straightened for a length of 8 miles. The cost of the canal was $3,500,000.

ANCHOR-DREDGING for maintaining deep water was successfully tried recently at the site of the new floating dock at Algiers, La. A part of a sunken coal barge had been found under the dock and the divers were unable to remove the whole of it. As the dock had to be submerged to take the battleship "Illinois," Mr. Anderson, of the Maryland Steel Co., secured a very heavy anchor and attached it by a strong hawser to a powerful tug. This anchor was sunk outside of the shore line of the dock site and dragged riverward into deep water. It was then lifted, all debris removed and then sunk and dragged again on a parallel line. In this way—though with much expenditure of force—the remains of the coal barge and other obstacles were entirely removed. The anchor acted as a plow and left deep furrows in the sand and mud at each trip, and the bottom was thus practically levelled. The 50 ft. of deep water required for the official tests of this dock are now available, and the trial will soon take place.

THE CINCINNATI WATER-WORKS TUNNEL reletting, involving a portion of the tunnel between the new purification works and the city pumping station, has been enjoined pending the results of a taxpayers' suit. The old contract was held by the W. J. Gawne Co. Gas was encountered, the work stopped, and finally the contract cancelled because the company did not carry on the work. The lowest bidder at the reletting was the Union Construction Co., of Milwaukee, at about $280,000. The only other bid was submitted by Frank H. Kirchner, of Cincinnati.

THE TORRESDALE SLOW SAND FILTRATION plant at Philadelphia is to be built by Daniel J. McNichol, of that city, for $3,734,000, and a clear-water basin will be built by the same contractor for $1,146,000. The contract for sand, terra cotta pipe and labor to complete the upper and lower Roxborough filter beds has also been awarded to Mr. McNichol at $627,000. The new Oak Lane reservoir will be built by R. A. Malone & Co., of Lancaster, Pa.

DAMAGE SUITS for nearly $3,000,000 for polluting the Pasaic River with sewage have been filed against the city of Paterson, N. J., by 44 riparian owners on the lower Passaic River.

A LARGE IRRIGATION CANAL in the Louisiana rice belt, with a total length of 30 miles and designed to water 100,000 acres of land near Vinton, La., will be constructed by a company of which Mr. W. L. Peters, of Vinton, is secretary. The head of the construction company is Mr. S. Robbins, himself a contractor.

THE WIDENING OF THE CHICAGO RIVER from Lake St. to Van Buren St., at an estimated cost of $1,438,310, has been decided upon by the Board of Trustees of the Sanitary District of Chicago. The work was decided upon at the meeting of the Board held on Dec. 30, 1901. According to the plans the river between the points named is to be enlarged to a uniform width of 200 ft. by removing a strip of land from 15 to 60 ft. wide along the north bank. The estimated cost of the improvement is itemized as folows.

41,156 square feet of real estate at $10 a foot..	$411,560
41,000 cubic yards of dredging at 20 cents a yard.	8,200
3,457 lineal feet of concrete dock at $150 a lineal foot	518,550
Estimate of consequential damages to property involved	500,000
Total	$1,438,310

Of the 41,156 sq. ft. of land which will have to be condemned in making the improvement about 20,000 sq. ft. are owned by the Pennsylvania Co., and are now occupied by the yard tracks and freight houses. It is expected that the railway company will make a vigorous legal fight against the condemnation of its property. The Drainage Trustees have formally ordered the legal department of the District to proceed at once with the condemnation proceedings.

ELECTRIC TRACTION is to be adopted for the narrow gage branch of the Chicago, Burlington & Quincy R. R. from Deadwood to Lead, So. Dak., a distance of 3.3 miles. In connection with this the line will be extended through the city of Lead as a street railway. The Arnold Electric Power Station Co., of Chicago, has the contract for the design and construction of the power plant, overhead wiring, and all work necessary for the operation of the line as an electric railway.

PRIZES FOR BEST AUTOMATIC COUPLERS for railway carriages, are offered by the Congress of Representatives of Russian Railways, Nevsky 30, St. Petersburg, Russia. The prizes are 1,000, 3,000 and 5,000 roubles, with the rouble worth about 51½ cts. in U. S. gold. Foreigners as well as Russians are invited to compete, and the conditions are as follows: Plans must be sent to the above Congress, at the address given, before April 15, 1903. They must be placed in sealed envelopes and inscribed "Competition for automatic coupling apparatus for railway carriages." In a separate sealed envelope, with some mark on the outside identifying it with the plans, must be placed the name and address of the sender.

A THREE-HINGED CONCRETE ARCH BRIDGE OVER THE DANUBE AT EHINGEN.

The government highway between the German cities of Stuttgart and Friedrichshafen crosses the Danube at Ehingen, in Wuerttemberg. A wooden truss-bridge at this point, built in 1848, had by 1897 become so dilapidated that the construction of a new bridge was imperative. In line with its general policy in these matters, the government resolved to erect a permanent structure of stone or concrete. As the neighborhood afforded no good building stone, but possessed extensive gravel beds, economy dictated the use of concrete as material for the bridge. Work on the bridge was at once proceeded with on this basis, and in a year, by the middle of October, 1898, the completed structure, shown in perspective in Fig. 1, was opened to traffic. The design and execution of the work offers a number of points of interest.

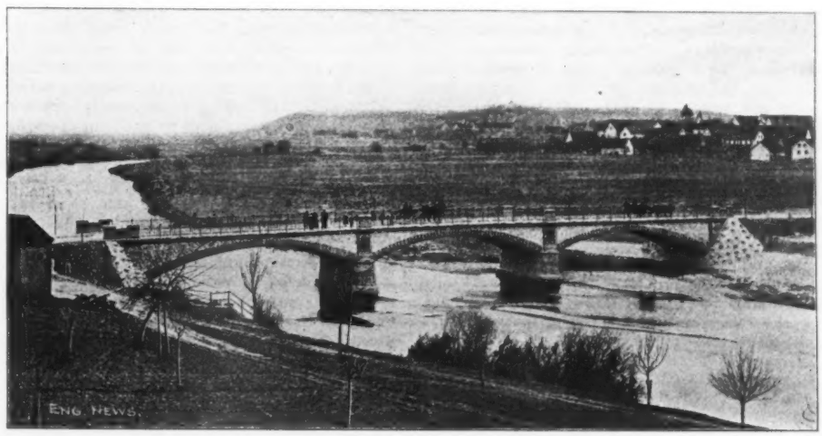


FIG. 1. GENERAL VIEW OF CONCRETE ARCH HIGHWAY BRIDGE NEAR EHINGEN, WUERTTEMBERG.

The site selected for the bridge was just above the old bridge, a temporary crossing being thus rendered unnecessary. The river bottom here was found to consist of gravel with pockets and layers of very compact sand to a depth of 10 to 14 ft. below low water, underlaid directly by bed rock. The mean depth of the river at low water is only about 6 to 8 ft. These favorable conditions as to bottom, the desired height of crossing, and the flow to be allowed for at high water determined the design of the bridge. The crossing is made in three spans of 69, 66 and 69 ft., with a rise for the center span of 7.2 ft. The arch rings are practically arcs of circles, and have a thickness of 2.30 ft. at the crown, 2.95 ft. at the springing lines at the river piers, and 3.11 ft. at the shore abutments. The springing lines here drop below those at the river piers. The effect will be noted in Fig. 1. The roadway is 25.3 ft. wide between handrails and 24.6 ft. face to face of arch; the carriageway has a width of 18 ft., and the footwalks respective widths of 4.6 ft. and 2.6 ft. The roadway of the two side spans rises toward the center with a grade of 1%. The central span is level. The live load used in the calculations was about 80 lbs. per sq. ft., in addition to a 15-ton steam roller. All three spans were constructed as three-hinged arches, leadplates about ¾-in. thick, constituting hinge joints, being inserted at the crown and the springing line. The whole structure is composed of concrete, except for granite cutwaters on the upstream face of the piers and limestone facing of the arch ring at the sides of the bridge. The arch rings are composed of ordinary concrete, but all other concrete has a fifth of its volume of rough stones embedded in it. The spandrel masonry is entirely separated from that of the abutments and the upper extension of the piers by vertical transverse slots running across the whole width of the structure down to the upper surface of the archring. Any motion of the arches due to temperature changes, settlement of supports, etc., is in this way prevented from producing unsightly cracks in the masonry of the finished bridge. The maximum stresses in the arch ring in lbs. per sq. in. are 220 at the crown, 185 at springing, 325 at joint of fracture, and 1,040 to 1,180 at the hingeplates. The piers are stressed to 75 lbs. per sq. in. at the springing lines of the arches, and 60 lbs. where they rest on the rock foundation.

Somewhat unusual methods were adopted in founding the abutments and piers. The depth of the layer of gravel and the large quantities of water flowing in it gave rise, in connection with previous experience in the gravel strata of that region, to the belief that the common method of excavating pits and resting the foundation masonry or concrete directly on the rock would offer difficulties on the point of excluding the water in the present case. In consequence of this belief, experiments were made as to the feasibility of converting the water-bearing gravel into a mass of concrete in place by the injection of thin cement grout. Twelve-foot lengths of 1½-in. pipe, with an iron driving point loosely inserted in the lower end, were driven down to rock, and then, by raising a few inches, lifted clear of the driving point. Cement grout was then pumped in until rapid rise in pressure indicated saturation; the pipe was then drawn up a small distance, grout pumped in again to saturation, and so on. The pipes were driven down at intervals of 18 to 20 ins. Subsequent excavations were made for the purpose of determining the success of the cementing. They showed that the distance to which the cement had flowed varied greatly with the amount of sand contained in the gravel. Where the gravel held but little sand, the cement was in some cases found at a distance of several meters from the pipe. In compact sand layers the penetration of the cement was practically zero. The cement everywhere had set in a satisfactory manner.

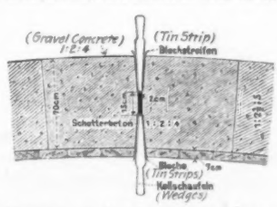


Fig. 2. Method of Closing Crown, Concrete Arch Bridge.

In accordance with these results, the left abutment was founded on a mass of concrete formed in place by such injection of cement. After completing the foundation, several holes were drilled through down to bed rock, to test the penetration and setting of the cement. The drilling showed

still if "precedent" had been waited on. In this case, however, there is no lack of precedent.

Again, if "precedent" is to be sought to solve the question it should be used negatively; if dams of over 100 ft. in height had been properly built and had failed, a precedent would then have been established as against further efforts.

Had the test wells been made in the Middle Branch embankment 15 years ago; had the same results been obtained as now, followed by the same conclusions; had these conclusions been accepted as we are now asked to accept them; then it is self-evident that the Carmel, Bog Brook and Titicus dams would never have been built, at least on their present lines; still they stand to-day as masterful examples of good construction and contradictions to the theories and conclusions of the experts, and it is so with many other earth dams with no flatter slopes than are proposed for the New Croton.

As regards the results noted at the test pits, I cannot see wherein they indicate conditions unfavorable as regards either filtration or the slopes produced by the action of free water.

While the water from Pit No. 6 found an outlet to the face of the core wall only a few feet distant, covering a very contracted portion of incomplete work, this should not be taken as a measure of the ultimate value of a completed embankment.

Mr. Sweet remarks further: "Mr. Fteley fails to distinguish between the slope assumed by water retained in an embankment, and that taken by water passing through an embankment made of material too porous to retain it; where the rule is clearly reversed and where the more porous the material the steeper the slope at which water will run through it at a given rate."

He also states: "The more compact the material of which the bank is built the steeper will be the slope of saturation." It is difficult to reconcile these two statements, so totally at variance with one another. The saturated portion of the bank is simply that portion below the inclined plane of the surface of the water in the bank whether the material is ever so coarse or ever so fine: porosity is merely a degree of compactness or vice versa, and all bank material will absorb water.

Slope implies motion in water, and there is no absolute retention of water in the outer bank of a dam having its base below the plane indicated by the loss of head in passing through the inner bank and then through a further obstruction of either masonry or puddle. It is simply a partial retention with motion through the bank governed entirely by a degree of porosity of the material, and when Mr. Sweet says "the more porous the material the steeper the slope at which water will run through it at a given rate," he properly reaches the same conclusion as stated by Mr. Fteley.

To satisfy the two conditions as advanced by Mr. Sweet, to wit: "The more compact the material the steeper will be the slope," and again, "the more porous the material the steeper will be the slope," seems impossible, though Mr. Sweet may be able to fix the line of compactness or porosity, on which the velocity depends, so that it will be consistent with both statements:

It has been suggested that there is a possible danger of movement of the slopes, in the nature of sliding on the underlying rock on the south side of the valley. Under different conditions than exist at the New Croton Dam such a movement might be feared. There, however, the rock is at such a great depth below the natural or restored surface that such a movement seems impossible.

Under the most favorable conditions for sliding a great bank, 150 ft. in height, has stood for a number of years with not the least obstruction to sliding other than its own inertia. When the refill is made the whole slope of the south side of the valley will be simply restored to its normal condition.

The rock slope on reaching under the valley flattens out and then rises again as it crosses the valley. Any movement then against this great abutting fill will not only necessitate a movement of the bulk of the entire mass, but an actual uplifting of this great body of earth which is 100 ft. in depth.

As to settlements in these banks: Levels taken on the banks of Bog Brook, Carmel Main and Auxiliary, and Titicus, before and after the reservoirs were filled, show only the Titicus inner bank to have settled. Long stakes were driven in these banks and careful levels were taken on them. The two banks at Carmel showed absolutely no movement, nor was any settlement found at Bog Brook. The inner bank at Titicus has settled over 18 ins. I am not conversant with the latest records, but that was for a period of four or five years. The settlement was vertical. This, of course, meant a displacement of so much water and a further compacting of material, and nothing more serious may be anticipated for the New Croton Dam.

At Titicus I attribute the settlement not to the fact that the bank was high, but it was not rolled with the same form of roller as at the Carmel Dam, and was not as perfectly compacted as it might have been. Banks well wet and rolled in thin layers will be stronger and more compact than most earths as naturally deposited. I doubt if a failure, due to filtration alone, of an earth dam, built with any care, can be pointed to. Those that have failed have failed from being overtopped by freshets, from insufficient provisions to carry off the surplus waters, clearly an error in design; from leaky or bursting pipes injudiciously built in an outer embankment; the burrowing through the banks by water animals, etc.; and in the latter cases a masonry core is the best preventative.

Some large masonry dams have failed also, with disastrous results.

Mr. John D. Van Buren (Engineering News, Dec. 26) takes a strong stand against the sufficiency of section of the masonry portion if the New Croton dam; and Mr. Gustav Lindenthal (Engineering News, Jan. 2) says it is not safe from the effects of earthquakes, and that masonry dams should be strengthened "with a network of steel buried in the masonry;" while the experts say the earth dam may be lacking in stability.

In view of these diverse opinions it seems that an earth dam may after all be the preferable form, and in many cases earth may be the only material available for construction.

An engineer of experience in such work has remarked that of materials used in the construction of dams earth is physically the least destructible of any. The other materials are all subject to more or less disintegration or change in one form or another, and in earth they reach their ultimate and most lasting form.

I believe there is nothing to be feared in either form if the work is carefully and properly done. And when a great piece of work is designed by, and brought nearly to completion under the supervision of, an experienced engineer, it should not be halted and changed at great expense unless it is conclusively shown to be wrong, and I claim that this the experts have not so proven.

Yours truly, Alfred Craven.

53 Highland Ave., Yonkers, N. Y., Jan. 6, 1902.

Notes and Queries.

J. E. B. sends us a copy of "The Practical Engineer," with the following marked in the department of "Questions and Answers:"

Editor "Practical Engineer:"

Kindly inform me whether or not a siphon will work where the short column is higher than from 28 to 34 ft. I have a shaft 50 ft. deep, and would like to know whether I can draw the water out to a greater depth than 34 ft. Respectfully yours,

John H. W. Schneider.

(Reply by the Editor.)—There is practically no limit to the height to which water may be lifted by the "siphon," provided you have a sufficient fall to overcome the friction caused by the lift. For instance, a siphon which is used to supply the city of Baltimore, Md., with water from the Gunpowder River has a lift of about 185 ft. over the top of a hill. All that is required to operate is a small pumping engine, in order to start the siphon to flowing. Once it is started, the flow is continuous so long as the mouth of the supply pipe continues unexposed to the air.

One is reminded of the oft-quoted saying: "It is better not to know so many things than to know so many things that are not so."

FAILURE OF DAMS NEAR ANDERSON, S. C., AND AT COLUMBUS, GA.

By B. H. Hardaway.

On the morning of Sunday, Dec 28, four masonry dams in the South failed. Two of these were across the Tallapoosa River in Alabama, near Tallassee,† one across the Seneca River in South Carolina, near Anderson, and one across the Chattahoochee River at Columbus, Ga. These dams have all been completed within the last year.

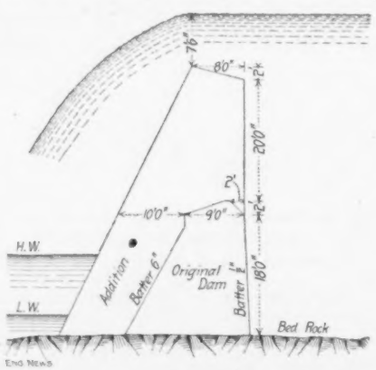


Fig. 1. Section of Masonry Dam Across the Seneca River, Near Anderson, S. C.

One of those at Tallassee was described fully in Engineering News of Dec. 5. The other was of about the same type. These two dams are reported to have had 10 ft. of water flowing over their spillways when failure occurred.

A sketch of the Anderson dam as it was originally built in 1897 and as is was raised in the fall of 1901 (or 1900?—Ed.) is shown as Fig. 1. The new part was anchor-bolted to the old. The original section has stood rises of 6 ft. on top of 7 ft. of wooden flash boards.

The failure occurred under a depth of 6 ft. on 4 ft. of flash boards, but the old portion of the dam withstood the flood.

The dam at Columbus (Fig. 2), owned by the Columbus Power Co., failed with a height of 10 ft. 2 ins. of water on the spillway, on the top of the wooden flash boards. The latter reached a point 3 ft. above the apex of the stonework and were anchored to the masonry with 1½-in. bolts 5 ft. long, placed in sets of three 5 ft. c. to c. along the length of the dam.

The maximum known rise in the Chattahoochee River would cause a depth of water on the spillway of 14 ft. The dam failed in two places. One gap about 200 ft. long was located on the Alabama side and one about 150 ft. in length on the Georgia side of the river, leaving an unbroken length of dam between these breaks of about 200 ft. denuded of flash boards.

In this unbroken part of the dam there were five openings 6 ft. wide by 8 ft. deep for sand gates, three of which were closed by the gates being down and two were open. On the part of the crest above these gates no flash boards had been placed, but the section at or near the gates had not been strengthened in any way. The dam failed to a depth varying from 5 to 20 ft. from the crest of the masonry.

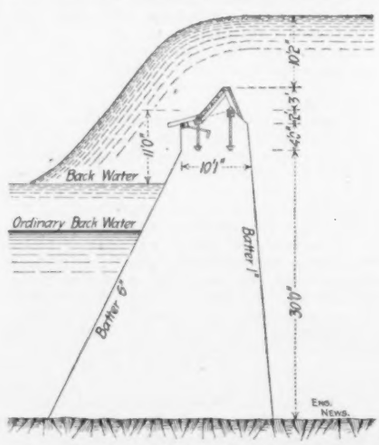


Fig. 2. Sketch of Columbus Power Co.'s Masonry Dam Across the Chattahoochee River at Columbus, Ga.

(Both these dams failed on Dec. 29, 1901.)

ELECTRIC TRACTION is to be tried on the Miami & Erie Canal, and the construction of the track and trolley line is now in progress between Cincinnati and Dayton, O., a distance of about 60 miles, of which 48 miles are now ready for the track. The system adopted provides for a Westinghouse electric locomotive, taking current from an overhead trolley wire and running on a track built along the tow-path. The canal is owned by the State. The Miami & Erie Canal Transportation Co., of Cincinnati, O., has the contract and lease for building the line for the entire distance between Cincinnati and Toledo, a distance of 244 miles, and has agreed to finish the work in about three years. The consideration is the payment of tolls to the State for each boat, the same as for boats hauled by mules. The work is being done under the general supervision of Mr. Charles E. Perkins, of Columbus, O., Chief Engineer of the Public Works of Ohio.

THE U. S. FLOATING DOCK, at Algiers, La., was successfully tested on Jan. 6. The battleship "Illinois" was lifted entirely out of the water in 1 hour 57 minutes, whereas the contract allowed 2 hours 40 minutes for this work. All of the machinery worked satisfactorily.

THE KONIGSBERG CANAL, commenced in 1890, has been opened to traffic. This canal permits large ocean vessels to reach Konigsberg, on the River Pregel, from the Gulf of Dantzig, and to pass the Frische-Haff, which formerly compelled them to unload on lighters at Pillau.

*Engineer and Contractor, Columbus, Ga.

†One of the Tallassee River Dams was not completed; see news note on the first page of this issue—Ed.

either good concrete or solid masses of cement, except for a layer of hard sand up to two feet in thickness. This sand, however, was so hard that loads up to 1,000 lbs. per sq. in. made no impression on it, and it was allowed to remain in the foundation.

To found the two river piers, cofferdams of sheet piling were driven to rock and made watertight by injection of cement through pipes driven around them. In the case of one of the piers, pipes were driven inside as well as outside, with the result that nearly the whole mass in the interior of the cofferdam was cemented into a block of concrete. On account of some layers of sand, however, about one-half of this mass was broken out again and the pier regularly built up of concrete above the remaining conglomerate. At the other pier, the cementing was carried out only around the outside of the cofferdam, making it perfectly tight. The interior was then excavated and the concrete of the pier built up directly on the rock.* At the right abutment a clay soil was met with, and the excavation to rock was carried on directly.

The cement used in the construction of the bridge was Portland cement of local manufacture, bought and tested by the government, and furnished to the contractor according to his needs. Each shipment of cement was tested for fineness, time of setting, and tensile strength. A sieve of 5,000 meshes per sq. cm. (=32,250 per sq. in.) was used, and 15% residue for arch ring cement, 25% for all other, fixed as maximum. All cement furnished satisfied the requirements and was accepted. The tensile strength of 1 to 3 mortar after seven days was uniformly 285 to 325 lbs. per sq. in. The proportions of the concrete used in the different parts of the structure differed considerably. In the foundation work of the river piers, 1 to 4 to 8 was used; the archring concrete was mixed 1 to 2½ to 5, and all other concrete 1 to 3 to 6. A yellow cement mortar was used to face the surfaces exposed to view except where, as at the faces of the arch ring and the cutwaters of the river piers, ashlar quoins and facing stones were used. The concrete was throughout used quite wet and soft and rammed well in 12-in. layers. In all portions except the archring concrete, stone rubble to the amount of 1-5 of the total mass was embedded in the work. The wet concrete bonded well with these stones and assured the uniting of adjoining sections of work better than the usual dry mixture.

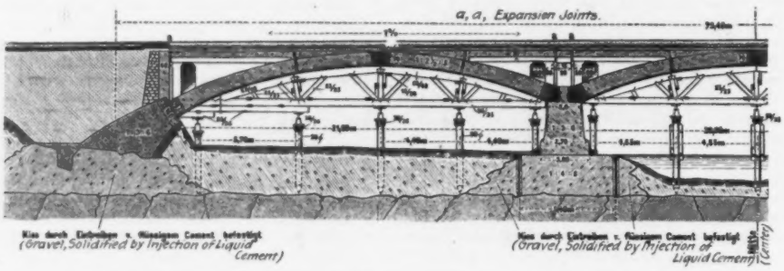


FIG. 3. HALF LONGITUDINAL SECTION, CONCRETE ARCH BRIDGE.

The arches were built each in two longitudinal sections, kept 30 cm. (= 12 ins.) apart. This space was filled with concrete after the two sections were completed, and to conceal the irregularities certain to show with a continuous intrados this portion was formed into an ornamental molded ring course with a projection of about two inches. The half sections of one side of the three arches were concreted in seven days. Work on each span was begun at the abutments and the quarter points simultaneously, working from the latter points to the key. Transverse sections about as wide along the bridge as the thickness of the archring were formed as separate sections with their adjoining faces as nearly as might be radial to the arch. When the crown was reached, the lead plates, shown in Fig. 2, were supported on a wedge-shaped mold-board and the keyblocks formed up of concrete; a similar mold-board formed the opening above the lead hinge. Plates of tin were set around these mold-boards as shown to facilitate their withdrawal. They were withdrawn after 24 hours.

Centers were struck four weeks after closing the arch. The wooden centers rested at their joints directly on jackscrews carried by supporting piles driven to rock and well braced in all directions. The construction is clearly shown in Fig. 3. To avoid settling by shrinkage of the woodwork while the concrete was still soft, the centers were kept wet by playing a hose on them during two weeks after closure of the arch. After the whole dead load was on the arch, the total sinking at crown amounted on the average to one inch, of which about one-third occurred in striking of centers. When the bridge was completed to this point, the wedge-shaped spaces above and below the lead hinges were filled with cement mortar.

The total cost of the bridge was about $21,000, of which about a third is to be charged to the approaches. For the above data in regard to the bridge, as well as the cuts reproduced, we are indebted to the "Centralblatt der Bauverwaltung."

*Similar methods of consolidating the subsoil were used in work on two other Danube bridges in the neighboring kingdom of Bavaria in 1898 and 1900. In both these cases it was employed to surround iron piles with cement or concrete as a protection against rusting. In the case of one of these bridges, at Laupheim, the pipes were sunk by jetting and the grout simply poured into the tops of the pipes with complete success.

A NEW DRY DOCK AT BALTIMORE, MD.

One of the largest dry docks in this country, capable of docking a vessel of 600 ft. in length and 70 ft. beam, with a draft of 22½ ft., has been lately completed at Baltimore, Md., for the Wm. Skinner & Sons Shipbuilding & Dry Dock Co. The following description of this dock is taken from information furnished to us by the above company, and from a paper by Mr. James Ritchie, M. Am. Soc. C. E., designer of the dock, the paper being one read before the Civil Engineers' Club of Cleveland and published in the "Journal of the Association of Engineering Societies" for October.

The principal dimensions of this dock, as completed, are as follows: Length over all, 628 ft.; width on floor, 62 ft.; width on top of keel blocks, 69 ft.; width on top of dock, 125 ft.; entrance at bottom, 60 ft.; entrance at top, 80 ft.; depth of water on sill at low water, 22½ ft.; depth at high water, 25 ft. Briefly described, this dock is built of piling and timber construction in the basin portion, with an entrance and power house combined of concrete masonry faced with Port Deposit granite in the entrance proper.

The dock is so located that the entrance had to be built 208 ft. out from the shore line, but within the established harbor lines; and an excavation was required extending 36 ft. below low water and 44 ft. wide by 164 ft. long. The foundation soil was fortunately a hard, compact white clay, and in this clay was excavated a cut-off wall 3 ft. wide and 3 ft. deep, extending clear across the entrance and its abutments. This was afterwards filled with concrete to prevent a leakage of water between the concrete and the clay at this point. To permanently protect this projecting end of the dock heavy bulkheads were built on either side, each consisting of 10 × 12-in. tongued and grooved sheet-piling, from 34 to 48 ft. long, driven in two rows 16 ft. apart and braced by guide piles, 12 × 12-in. timbers and 1¾-in. iron rods spaced 10 ft. apart; the inner faces of these bulkheads were from 132 to 145 ft. apart. On the south side of the dock a third row of sheet-piling was driven, and between the two outer rows the concrete of the power house was laid.

The entire area of the dock-basin was excavated

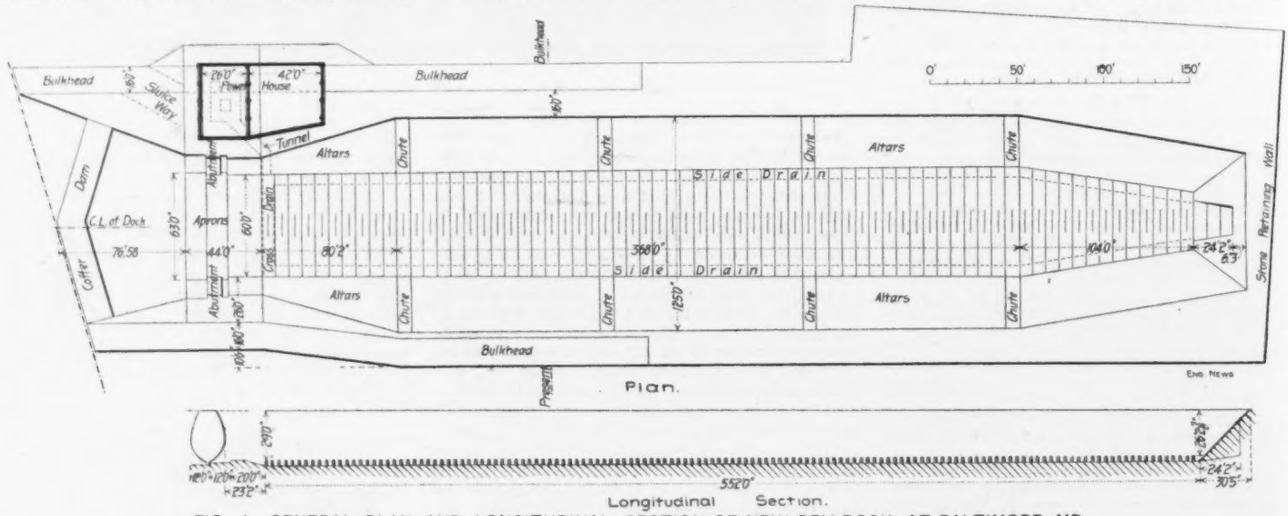


FIG. 1. GENERAL PLAN AND LONGITUDINAL SECTION OF NEW DRY DOCK AT BALTIMORE, MD.
Built for the Wm. Skinner & Sons Shipbuilding and Dry Dock Co., Baltimore, Md.
Ritchie & Ruple, Cleveland, O., Designing and Superintending Engineers. Delaware Construction Co., Wilmington, Del., Contractors.

by dredging to the depth required and a coffer-dam was built to close the entrance opening between the bulkheads. The dredged portion was next pumped out, and as this was done the two bulkheads were held against the outside pressure by heavy cross-bracing supported on lines of temporary piles. Two rows of 10 × 12-in. tongued and grooved sheet-piling were also driven at each end of the proposed excavations for the entrance masonry.

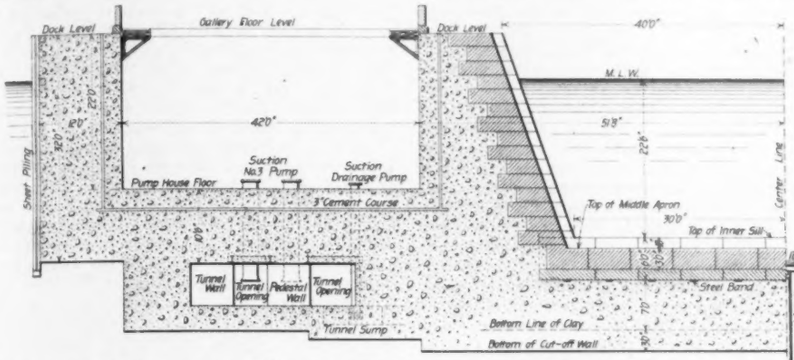


FIG. 2. HALF-SECTION AT ENTRANCE OF DOCK SHOWING POWER HOUSE AND TUNNEL.

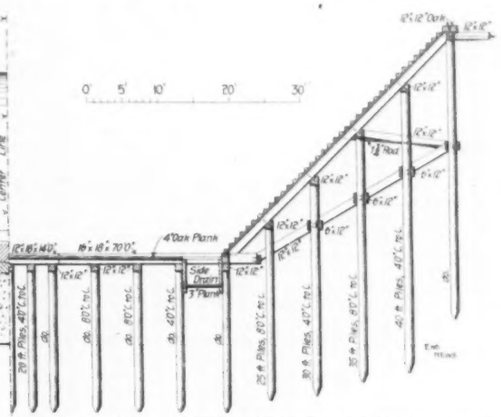


FIG. 3. HALF MIDSHIP SECTION OF DOCK-BASIN.

Above the hard clay mentioned was fine white sand in pockets, and above this sand was a stratum of hard gravel in which some of the sheet piling had stopped. These conditions caused considerable trouble from water, which came through the fine sand, and careful handling of the concrete was necessary. It should have been mentioned that the sheet-piling of 10 × 12-in. stuff was continued in a single line from the bulkheads mentioned to the east line of Belt St., where it meets a stone retaining wall.

The power house is located in the south abutment of the entrance works, and the tunnel to convey water from the dock to the pumps is formed in the concrete base. This tunnel is 6 ft. square, with a concrete roof 10½ ft. thick. In the walls of the tunnel and in the roof 12-in. iron beams are embedded, and the suction pipes and the foundation bolts of the engines are also built into the concrete, the suction pipes having flanges to cut off water seepage. The walls of the engine-room are concrete, 11 to 16 ft. thick, and in the walls and floor is placed a 3-in. cut-off layer of Portland cement mortar. This concrete work is extended across the entrance; and to protect the side slopes at the head of the dock concrete filling was placed back of the altars for a distance of about 60 ft. from the head and at the point where the timber and masonry work joins a wall of concrete is built back of the slopes. All of this concrete was made of 1 part "Dragon" American Portland cement, 2 parts sand and 5 parts of broken stone.

The bottom and slopes of the basin are supported by piles, arranged as shown on the midship section and capped longitudinally by 12 × 12-in. timbers. The floor cross-timbers are made of Oregon pine, 16 × 18 ins. and spaced 4 ft. apart. Every alternate floor timber is 70 ft. long the others 14 ft. long, and all of these timbers are securely drift-bolted to the piles and to each other. The 12 × 14-in. slope-timbers are framed into the top of the long floor-timbers, and between each main slope-timber is an intermediate 8 × 14-in. slope, framed into a filler, which is also framed into the sides of the long floor-timbers.

The floor is made of 4-in. oak plank, spiked to joists bolted to the longitudinal capping on the piles. The surface of this floor slopes from the center to each side and thus meets the side drains of the dock, which latter empty into a main cross drain leading to the tunnel under the pumps. The five rows of piles supporting the slopes are braced by 12 × 12-in. timbers and by waling pieces, as shown. The altar timbers, or steps, have a rise of 10 ins. and are all securely spiked to the slope timbers; the top of the slope is finished by two 12 × 12-in. timbers bolted together and drift-bolted to the upper cap. In these slopes, on each side, are built four chutes for delivering material; these extend from the bottom to the top, and each chute is 8 ft. wide and is made of 6-in. oak plank, with sides made of the 8 × 14-in. timbers. The keel-blocks, placed 4 ft. apart on the center line of the dock, are 3½ ft. high and are made of three pieces of 12 × 16-in. and one piece of 6 × 16-in. oak, bolted to the floor cross-timbers. The slide, or bilge-blocks, are placed every 8 ft., and they can be adjusted to the shape of the vessel both by hinging the upper block and by sliding the entire block on the floor-timbers.

The power plant, located in the south abutment, includes three 350-HP. water-tube boilers, three 36-in. centrifugal pumps with a total capacity of 105,000 gallons per minute and capable of emptying the dock in 1½ hours, and one 14-in. centrifugal pump for taking care of any leakage. The main boiler plant furnishes steam for the main pumps, but there is an auxiliary tubular boiler for supplying the drainage and caisson pumps, the draft fan, fire pumps and dock-washing pumps. These two fire and washing pumps are connected with water pipes running the full length of each side of the dock, connected with the city water service and provided with hose connections between every chute. The pumps and engines were built by the Morris Machine Works, of Baldwinsville, N. Y., and the boilers by the Heine Safety Boiler Co., of St. Louis, Mo. The steam and water piping was done by Wallace Stebbins & Sons, of Baltimore, Md.

The entrance gate is a steel caisson set in the grooves of the granite facing of the entrance and giving an opening of 75 ft. 8 ins. at the low water line, with 22½ ft. of water on the sill. In this

Fig. 4. General View of Interior of Completed Dock; Looking Towards Entrance.

caisson are six 36-in. openings fitted with Ludlow gate valves for filling the dock. On the main deck of the caisson are located the engine and pump for emptying and floating the caisson gate, steam being supplied by a hose connection with the auxiliary boiler referred to. The entrance proper and the floor of the aprons are faced with Port Deposit granite, finely cut. The sill stones are 6 ft. thick and weigh from 14 to 17 tons each, and under these is concrete 8 to 10 ft. thick. The apron floor is from 2 to 3 ft. thick. The granite courses in the abutments are 2 ft. high, the stretchers being not less than 6 × 4 ft. in plan, and the headers 3 × 6 ft. All these stones are doweled together on the beds, and the outer sills are tied to the inner ones by steel bands embedded in the concrete under them. The steel caisson gate was built by the Spedden Shipbuilding Co., of Baltimore, Md.

The plans and specifications for the building of this dock were prepared, and the work of construction superintended by Mr. James Ritchie, M. Am. Soc. C. E., and Mr. C. P. Ruple, forming the firm of Ritchie & Ruple, civil and constructing engineers, of Cleveland, O. The specifications and general plans for the pumping plant were prepared by Mr. C. L. Reeder, Mechanical Engineer, of Baltimore, Md. The Delaware Construction Co., of Wilmington, Del., executed the excavation, piling, masonry and timber work, and C. H. Fath & Son, of Cleveland, O., built the power house superstructure.

BLAST FURNACE EXPLOSIONS.*

By John M. Hartman.†

In the summer of 1876, while the writer was in temporary charge of the Durham furnace, an explosion occurred that threw a bell and hopper and the stock out of the furnace, scattering the stock over the buildings and ground. The pressure of 11 lbs., then considered high, blew too hard on the cinder notch, causing the keeper to slack blast for three seconds to bot it up and then throw it on, when the explosion occurred. As usual more or less with all anthracite furnaces, there was a scaffold in it, but this happening to be a large one caused a larger explosion. The furnace was 19 ft. × 76 ft., a large one for that day.

The pipe stoves were attached to a receiver of double the usual size. The pressure lowered to 4 lbs. after the scaffold fell. All furnaces have above the zone of fusion a zone of reduction, filled with disintegrated ore, limestone and fuel. Between the interstices of the fuel, ore and stone, the finely divided carbon fills up the space, forming almost a solid zone, through which the gas has to force tis way upward. This finely divided carbon mostly comes from the decomposition of carbon oxide, and in this finely divided state, when red hot, is extremely explosive.

Taking a modern blast furnace, say 20 ft. × 90 ft., its cubic contents will be 20,000 ft. Blowing, say, 30,000 ft. per minute through the tuyeres, the furnace would be filled with air in 40 seconds from the engine. Suppose the scaffold in this modern furnace were up 18 ft., the fuel being burned away below the scaffold. There would then be free space between stock in the furnace and the space below the scaffold amounting to 4,500 cu. ft., which the engines would fill in 9 seconds. Taking 12 lbs. air pressure against the scaffold, suppose it gives way, the air pressure is then released to, say, 4 lbs., when the slip occurs. Immediately the volume of air back to the blowing engine expands and the engines go quicker, the air rushes into the furnace, and not having a large body of fuel at hand to take it up and convert it to gas, free air gets into the finely divided carbon, while the mass is falling. The carbon explodes and the furnace becomes a huge cannon, shooting the stock out and scattering it in all directions, causing loss of life and destroying the buildings.

Taking the 20-ft. furnace with 30,000 ft. of air, and supposing it to scaffold suddenly, it takes 4½ hours to burn out all the coke 18 ft. up to the scaffold, leaving a large cavity to be filled with the scaffold and stock when it drops. Quick as the stock stops settling at the top the furnace commences to scaffold. If in one hour's time the furnace does not settle, the founder must make it settle by cutting off the blast or by using dynamite. A founder will find that with a given volume of air the scaf-

*From "The Iron Trade Review."
†1225 North Front St., Philadelphia, Pa

fold will commence at a certain distance up. He can have a hole in the side of the furnace at that point. Opening this hole, he can drive in a piece of gas pipe to the center, then insert the cartridge and shoot, which brings down the scaffold. It is rarely that a second shot is required.

There is a direct connection between the top and bottom of the furnace in operation. For every 30,000 ft. of air entering per minute at the tuyeres, there must be 445 lbs. coke entering at the top. If less coke enters, the furnace is hanging up and the deficiency must be made good when the furnace drops. After the furnace hangs up for an hour, the cavity burned in the coke becomes large and dangerous. It must be corrected at once, as the loss of life must not be risked nor the loss of property.

Since 1876 there have been a number of explosions in the boshes of furnaces, throwing out the bells, hoppers and shooting out the stock—some of them attended with loss of life. This form of explosion must not be confounded with a gas explosion at the top, which throws out the bell and hopper but no stock. When these bosh explosions occur and the men are near the top, the finely divided carbon falls on them and sticks fast, burning off the skin and causing great pain. In these days of fast driving the constant care of the furnace by one reliable man has been lost sight of. If we wish to avoid loss of life and property we must have him.

At a charcoal furnace some years ago the founder attempted to stop the gas rushing out at the open top and endangering the buildings while blowing out. He filled the top down to the flues with charcoal braize well rammed in, which formed a good plug. The furnace was blown down and a hole knocked in above the tymp, when the air rushed in, exploded the gas in the furnace, which blew out the plug at the top, scattering it in a burning condition over the buildings and burning them down. The furnace became a retort and gasometer, waiting for the air to explode it.

It is peculiar that in these days of numerous engineering colleges there is not a chair devoted to blast furnace practice. In no department of metallurgy is there so little known and written of as the blast furnace, and in no department is there higher skill required in chemistry, mechanics and physics than in this one. In the Bessemer and open-hearth practice the material is at hand—iron and fuel; manipulation only is required. With the blast furnace the different materials are of great diversity, widely scattered, must be properly proportioned and the furnace must be properly handled to get a ton of pig of the exact character required.

To get the ton of pig the founder has to use, say, four tons of material of widely different qualities, while the ton of steel does not require over two tons. With our present fast driving, it has become necessary that something be done to stop the loss of life in modern furnace practice.

If young men who are to run blast furnaces are properly taught of the dangers of the furnace and better ways of handling them, the iron industry will be well repaid for the expense of the chair, but the professor must be a man who can better use a pair of hand leathers than handle kid gloves.

RAILWAY EMPLOYEES IN THE UNITED STATES.

The November Bulletin of the Department of Labor contains an interesting article on the above head, contributed by Samuel McCune Lindsay, Ph.D., of the University of Pennsylvania. From it the following notes are taken:

On June 30, 1900, there were 1,017,653 employees of all classes engaged in railway transportation in the United States, or an average of 529 men per 100 miles of line. While this average is greater than in any previous year, and is only approached by the record of 1893, a comparison with foreign methods of management show that in 1895, the railways of Great Britain employed five times as many men per 100 miles of line as did American roads for the same year; France had nearly 2½ times as many, and Prussia over 3½ times as many. The varying intensity of traffic in various sections of this country is shown by the fact that for the Middle Atlantic group of States the above average is 1,140 men per 100 miles; in the New England States it is 827; and the lowest average is 303 men in the Middle Northwestern group of States.

The qualifications demanded in the chief grades of railway service are relatively high. In grades of service employing 388,666 men, or 38% of the grand total of railway employees, it is usual to require the applicant to fill out blanks, giving general information as to his age, previous experience, positions held, references, etc. This blank must be accompanied by a certificate of good character, and the applicant must submit to a rigid physical examination. On one system operating 5,000 miles of line and employing 32,000 men, these blanks are especially minute in these questions: referring to use of intoxicants and drugs, eyesight and hearing, cause of dismissal from other service, previous application for service and previous physical examination, details of any previous injury received, litigation with any railway company, etc.

Of the 62% of employees in the United States not subjected to this close investigation, the general officers, office clerks and skilled mechanics cover 23%, and 39% of the grand total are ordinary unskilled laborers, for whom the sole qualifications are physical health and the willingness and ability to perform manual labor. For all except the latter class, a good common-school education is demanded as a basis. As a rule, the important railways require the major part of their employees to work up through the lower grades of service, and college graduates frequently begin on train service as firemen on freight engines. It is admitted that the better the general training and education a man has before entering railway employment, the more rapid is apt to be his advancement. The age qualification is lower than it was ten years ago, and it is now difficult for any one over 35 years of age to enter railway service without previous railway experience; and some roads do not employ new men at all over this age.

Under existing conditions any statement of average wages received for railway service would be perplexing and misleading, and the same may be said of average hours of labor. As a general rule wages in all classes, from general officers to laborers, are now higher than they were in 1892, the increase being gradual since then. The Bulletin, however, presents a large amount of tabular matter on these heads to which those interested are referred.

In this country the railway service is very generally looked upon as a life work. Entrance to it is, as a rule, only open to young men, who have time to work up from the lower ranks. In late years there has been a great change in the rules governing discharge on leading railways. Boards of inquiry, to which appeal may be taken in case of discharge, are very common; and the number of causes which permit summary dismissal have been greatly reduced. In making promotions most railways apply very strict civil-service methods, and sentiment and politics play a very small part. Promotions are usually made within the service, and transfers from one road to another, except among officials of higher rank, are exceptional.

Pension systems are now in force on several of the great railway systems, and relief and insurance societies are common among the railway brotherhoods. As an example of the latter the Locomotive Engineers' Mutual Life and Accident Insurance Association, in 1890, had a membership of 35,000 men; it paid $810,750 in total claims that year, on a personal assessment of $17.34 on $1,000, and had outstanding a total insurance of $53,714,250. The Brotherhood of Locomotive Firemen, in the same year, had 36,084 members; paid $458,672 in claims, and had $49,312,500 outstanding in insurance.

The six railways having relief departments are: The Pennsylvania R.R., the Chicago, Burlington & Quincy, the Philadelphia & Reading, the Baltimore & Ohio, the Pennsylvania Lines West of Pittsburg and the Atlantic Coast Line. These relief departments are in addition to pension departments established by these and other companies, and apply especially to cases of sickness and temporary disability; the funds are raised by voluntary contributions from employees and by contributions by the companies. The Bulletin goes fully into the working of these pension and relief departments.

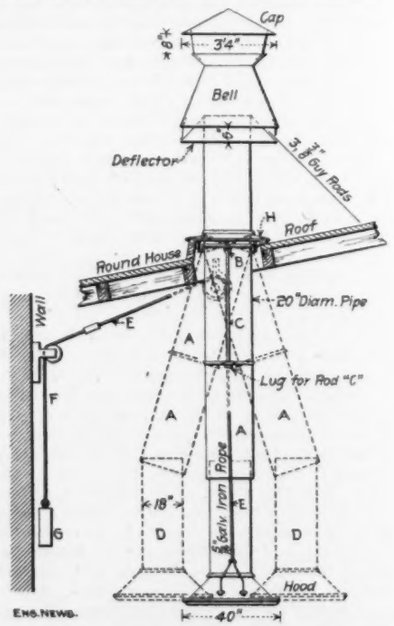
In the direction of educational efforts to improve the railway service European roads do more than American roads. Very few of the latter furnish directly any technical education for their employees, as is quite general on European systems. The Young Men's Christian Association, with a total membership of 37,000 railway employees, is substantially encouraged by railway corporations; and a number of their branch associations have educational programs which more or less meet the particular needs of railway employment.

Several railway companies have established saving funds, to encourage thrift and to provide a safe and easy method for their employees to invest their savings. One founded by the Baltimore & Ohio R. R. Co. in 1882, has had, to June 30, 1900, total deposits of $4,000,000; and nearly $3,000,000 of this was loaned to employees at 6% and used in building and buying houses, etc. The fund has paid 5½% interest to depositors for several years. In 1900 the Employees Saving Fund of the Pennsylvania R. R. Co., founded in 1887, showed a credit of $2,717,709, with 6,52[illegible] depositors. The Illinois Central R. R. Co., in 1896, started a plan by which the employees could purchase the stock of the company by payments of $5, or multiples of that sum; on these deposits 4% interest was paid until the stock was secured—provided the purchaser did not allow 12 consecutive months to elapse without making some payment. On June 30, 1900, 3,090 shares of stock had thus been purchased and paid for, at an average cost to the purchaser of $98.13 per share—while on that date the market price of the stock was $116 per share. The price of this stock, as sold to employees was fixed by the company on the first of each month; presumedly based on the market price, as the rise in value caused a falling off in the purchases by the men.

SWINGING SMOKEJACK FOR ROUNDHOUSES.

The majority of modern roundhouses are now equipped with swinging and extensible smokejacks, which can be adjusted to the smokestacks of the locomotives, as the old style of fixed smokejack was not sufficient for proper ventilation of the house. The accompanying illustration shows the Roe smokejack, which is used by a number of railways and is being fitted to the roundhouse of the new switching yards of the Chicago Transfer & Clearing Ry.

The smokejack is of cast iron, 20 ins. diameter and ¼-in. thick, made in 5-ft. lengths, A, A, put together with watertight joints. The pipe swings on a 1¼-in. cast-iron pivot B, through the flange



The Roe Swinging Smokejack for Roundhouses. Railway Supply Foundry Co., Chicago, Makers.

of the iron roof collar, and to this bolt are attached two ¾-in. suspension rods C, which support the lower section of the pipe by nuts screwed up against lugs on the joint flanges of the pipe. The bottom section of the jack, D, is an 18-in. pipe, 48 ins. long, sliding within the end of the 20-in. pipe, and giving a telescopic adjustment of 42 ins. to fit the height of different engines. It is raised or lowered as required by means of two 5-16-in. galvanized iron ropes E, attached to the 40-in. hood

and carried over sheaves in the roof; the two ropes connect with a single rope, F, which is led over a sheave on the wall, and has a counterweight G attached to its end. As the pipe is able to swing, it will not be broken or torn down if an engine should be started out of the house before the drop hood is raised, and the drawing shows the extent to which the pipe can swing.

The roof opening is covered by a cap ring, with an oval flange inside, 8 ins. deep, to which the swinging pipe is pivoted, while to the upper part is fitted the outside pipe, the top of which is surmounted by a deflector, bell and cap. The height of the outside portion and the length of the inside pipe can be made to suit any requirements.

This smokejack is manufactured by the Railway Supply Foundry Co., 355 Dearborn St., Chicago, to which company we are indebted for drawings and other information.

EXPERTS' REPORT ON SEWAGE DISPOSAL FOR THE LOWER PASSAIC VALLEY.

After years of agitation a plan for stopping the pollution of the Passaic River by the sewage of Paterson, Passaic and the intervening New Jersey towns, has been devised, which bids fair to win the approval of the communities involved and to be put into execution. An unusually dry summer in 1895, with a consequent low river stage, brought home to the people of the Passaic valley the fact that the once beautiful river flowing by their doors had become little better than an open sewer. Public meetings were held, committees appointed, and an appeal made to the legislature. As a result the State legislature of 1896 provided for the appointment of an investigating commission. In February, 1897, the commission submitted, with its approval, a report by Messrs. A. Fteley, M. Am. Soc. C. E., of New York, and the late Chas. E. A. Jacobson, recommending a trunk sewer discharging into Newark Bay (see Engineering News, March 11, 1897). The proposal to discharge crude sewage into Newark Bay met with a storm of protest from Bayonne, which lies opposite the outlet then proposed. At the other end of the proposed trunk sewer Paterson objected to the whole scheme on account of cost. The various communities involved were more or less distrustful of each other and of the legislature, each fearing that it would be called upon to bear more than its share of the expense of any plan of disposal, and that it would not be properly represented in the commission to whom construction would be entrusted. Consequently nothing definite was accomplished from 1897 to early in 1901, except to appoint more investigating commissions, and finally a State Sewerage Commission, and to authorize the formation of sewerage districts and the construction of sewage disposal works therefor within the State. The State Sewerage Commission was authorized to prohibit water pollution, and in 1900 it served notice on the offending municipalities that they must stop discharging sewage into the Passaic River by July, 1902.

Efforts to form a sewerage district during the fall of 1900 failed on account of the distrust already mentioned, and because no definite plan of disposal was under consideration. Meanwhile Passaic had become strongly inclined to independent action, with disposal by septic tanks and contact filter beds. Paterson was still opposed to the trunk sewer, and there was more or less talk of dividing the valley into two sections, each to solve the problem as best it might. The legislature of 1901 adjourned without giving any further aid, and a strong effort to secure a special session was futile. Finally, the governor authorized the State Sewerage Commission to expend $8,000, from funds under his control, in a further study of the problem. To this end Messrs. Colin R. Wise, of Passaic; Robert M. Watson, of Rutherford; and Wm. Ferguson, of Paterson, were engaged to devise plans for the upper district, and Mr. Jas. Owen, M. Am. Soc. C. E., of Newark, to submit a plan for the lower section. These plans, in turn, were submitted, on Oct. 10, 1901, to Messrs. Rudolph Hering, J. J. R. Croes, and Wm. M. Brown, Jr., Mems. Am. Soc. C. E., who submitted their report to the commission on Dec. 2. The report will soon be laid before the legislature, with the approval, we understand, of the commission, and with a recommendation that the commission be empowered to carry it out at the expense of the communities involved.

After preliminary statements, including reference to a consultation with Mr. H. W. Clark, Chemist of the Massachusetts State Board of Health, regarding septic tanks and contact filter beds, the experts' report outlines the problem before it as follows:

> The territory, the sewage of which flows into the Passaic River, comprises portions of Passaic, Essex, Bergen and Hudson counties, covering an area of about 53,200 acres. It embraces the cities of Paterson, Passaic, Newark, Orange and East Orange, and the towns of Montclair, Bloomfield, Harrison and Kearny; the whole or parts of the townships of Acquackanonck, Franklin, Union and Belleville, and part or all of the boroughs of Garfield, Wallington, Carlstadt, Rutherford, East Newark and Glen Ridge, and with a present population of 520,000 people.
>
> Paterson, Newark, Arlington, East Newark, Harrison and Kearny have systems of sewerage in operation, collecting the surface water as well as the sewage, and Passaic, Orange, Bloomfield, East Orange, Montclair, Belleville and Rutherford have sewers which collect the sewage only, leaving the surface water to follow the natural channels to the river.

The cheapest and altogether the most practicable manner of diverting sewage from the river, the engineers say, is by combined, rather than individual municipal action, particularly in view of the difficulty of finding sites for purification works in so populous a section of the country. It is likewise true that disposal by dilution, where practicable, is preferable to purification works, which "require extensive plants, costly to construct and maintain, which must necessarily be located at a distance from habitations."

THE PROPOSED TRUNK SEWER.

Having concluded that joint action, with a single trunk sewer, is the best plan, the location, grade and size of the sewer, and the necessary pumping stations are described substantially as follows:

> LOCATION.—We find that a trunk sewer can be constructed near the west bank of the Passaic River from the southerly line of the city of Paterson to near the southerly boundary of the city of Newark, which will convey the entire domestic and manufacturing wastes and a limited proportion of the storm water from the territory under consideration to the westerly shore of Newark Bay near the point where the Lehigh Valley R. R. crosses the bay.
>
> To secure the necessary gradient to collect the sewage of Paterson and the other contributing communities at such a velocity as to prevent the deposit of sediment, namely, 3 ft. per sec., when flowing half full, the inside top of the sewer must be 14 ft. above mean high tide at its upper end and 14 ft. below mean high tide at its lower end.
>
> SIZE.—The sewer must be 7 ft. in diameter at Paterson and 11½ ft. in diameter at Newark Bay. This will require the erection of a pumping plant at the shore of the bay and its constant operation to keep the sewage in motion. The length of the sewer between the above points will be approximately 82,900 ft., or 15.7 miles.
>
> In locating this sewer, it was found that a small district in the city of Passaic and adjoining townships lay at too low a level to discharge into it by gravity. This district of about 1,500 acres is 13 miles from the pumping station at the bay. To accommodate it one of three methods of construction must be adopted. Either the whole length of the sewer must be lowered 16 ft. and the entire quantity of water pumped that much higher at the terminal station, or, as suggested by Messrs. Fteley and Jacobson, a second pumping station can be located at Second River, to lift the sewage collected at that point, or the sewage from that small low level district alone may be pumped 16 ft. and discharged into the trunk sewer at Passaic. The latter method was adopted by us as the more economical one at the present time. The pumping station would be located at Wallington on the east side of the river, opposite the race.
>
> ROUTE AND GRADE.—The general route and grade recommended by us are such as appear most practicable and economical from the plans and profiles of Messrs. Fteley and Jacobson in 1897, and Messrs. Wise, Watson, Ferguson and Owen in 1901. To determine the best line for construction and exact estimates of cost, we find that more extended surveys and boring are necessary. The study of details may indicate a preference for a crossing of Newark Bay north instead of south of the Lehigh Valley R. R.
>
> In determining the dimensions of the trunk sewer we assume that its capacity shall be such as to provide for conveying easily all the sewage of the district for 30 years from the time of its construction, or until the year 1935, at which date the population contributing to it is estimated at 1,500,000.
>
> The quantity of domestic and factory wastes and ground water entering the sewer is assumed substantially at 150 gallons per day per head of population, in accordance with the suggestions of your engineers.
>
> The length of the sewer is such that the maximum of the discharge from Paterson, one of the largest contributors to the sewer on account of its great manufacturing wastes, will not reach the pumping station until after the maximum discharge of Newark has passed, so that the fluctuations in the quantity to be pumped hourly will not be very great. The capacity of the sewer is made large enough to convey 270,000,000 gallons of sewage per day at its lower end, to allow for fluctuations and for the first storm water which may enter the sewer where a combined system of sewerage is used. The latter is assumed by Mr. Owen as equivalent to 100 gallons per head per day. For an excess of flow in heavy rain falls, overflows to the river must be provided at proper and convenient points along the route.
>
> When in the future the increase of population and corresponding amount of sewage will diminish the capacity of the trunk sewer for storm water so that the overflowing amount is too foul to be any longer safely admitted in the river, its capacity for legitimate purposes may then be increased by a further restriction of the rainwater admitted to the sewers, as already done in other cities under similar conditions with beneficial effect, and the life of the proposed trunk sewer may thus be materially lengthened.

FINAL DISPOSAL.

Disposal into Newark Bay, even after partial purification of the sewage, is held to be impracticable for more than a few years, owing to insufficient dilution and the slowness with which the sewage would pass out of the bay. On the latter point the report says:

> Using the so-called piston method of flow, which supposes the water to move down the stream with equal velocities throughout the entire cross section of its bed, we find it would take 15 days of tidal oscillation for sewage, if discharged at Passaic Light, to reach a point in New York Bay from which it would not return into Newark Bay with the tide. In other words, the Newark Bay and Kill von Kull might contain the sewage of about 15 days during the summer season, when the river flow is lowest. If the sewage is discharged into the center of the channel, where the velocity is greatest, we may assume this period to be somewhat shortened.
>
> Even then and under otherwise favorable conditions, we could expect, for a population of 1,500,000, a dilution of sewage of perhaps only one part to five parts of clean water, which would not dilute the sewage sufficiently to prevent offensive conditions in the upper part of the bay.

Instead of attempting to purify the sewage so it might be discharged into the bay the report recommends that the outlet sewer be continued some five miles across Newark Bay, the Bayonne peninsula, and into deep water in the upper part of New York Bay, "where there is a large volume of water and a strong current, ensuring thorough dilution and inoffensive disposal."

> PUMPING STATION.—At the outset, three 50,000,000-gallon pumps should be provided, giving a reservoir capacity of 50,000,000 gallons a day. The station would be located as near as feasible to the shore, giving both water and rail connections, and facilitating overflows into Newark Bay. The pumps would deliver sewage into two 60-in. cast-iron force mains, laid in dredged trenches, which, with a total lift of 40 ft., would discharge the sewage at the Bayonne shore, on the east side of Newark Bay, into a gravity sewer 11½ ft. in diameter. The further characteristics of the outlet are given in the report as follows:
>
> This sewer would then approximately parallel and run just south of the Morris Canal, and would have a sufficient elevation to deliver the sewage through a single outfall pipe about 13,000 ft. in length, into Upper New York Bay. The outfall pipe would leave the shore at an elevation of about 20 ft. above high water, and extend about two miles into the bay, to a point north of Robbin's Reef ledge, practically in the center of the bay, in the vicinity of which the channel is 70 ft. in depth.

The report next gives the reasons for recommending disposal by dilution in New York Bay, instead of purification works on the Newark meadows and discharge of the effluent into New York Bay. Discussions of this sort are almost always interesting, and this one is particularly so at this time, since it reviews some features of septic tanks and bacterial or contact filter beds for treating large quantities of sewage. On these points the experts say:

> SEPTIC TANKS.—The only alternative is, as before stated, partial purification by works on the Newark meadows. A sufficient purification can be effected, as has been proposed, by septic tank treatment followed by contact beds, provided a sufficiently large plant were installed. The cost of this method, particularly when applied to large quantities of sewage, is still a matter of doubt, arising from a lack of sufficient experience with operation on a large scale.
>
> We are somewhat uncertain as to the quantity of sludge which will be deposited in the present case and require

mechanical removal. The length of time during which sewage of the character and strength here expected should be subjected to septic action ranges between ½ and ¾ day. There is also still some doubt as to the best preparation of the tank effluent for the contact beds, the life of the latter and the best details of operation.

"No bacteria have yet been discovered which will consume mineral matter," says Mr. Gilbert Fowler, the superintendent and chemist of the Manchester, England, Sewerage Works. "Grit chambers," or catch pits, he considers essential before passing the sewage into the septic tanks, the size of such catch pits depending upon local conditions.

"If the catch pits are too small all the grit will not be arrested; if too large, sludge, which might otherwise have been bacterialized, will be taken away with the grit." There being in the Passaic sewer district several square miles, which are now sewered on the combined system, and therefore admitting surface water, a large amount of grit will necessarily be conveyed to the pumping station, and this, to ensure proper septic action in the tanks, should be separated by deposition before entering the tanks.

The amount of sludge deposited in the tanks varies with the character of the sewage, and, according to Mr. Clark, would range in different cases from 30 to 80% of the total suspended matters, the higher figure being for very strong sewage. Under conditions to be expected in the Passaic Valley, he estimates that not less than 40% of the total suspended matter would be deposited in the tanks.

Part of this deposited matter would be liquefied by septic action, and part of it would have to be removed by mechanical means. The latter portion would approximate 30% of the total amount suspended in the original sewage, and, with a population of 600,000 people contributing to the sewers as existing in the Passaic Valley, upwards of 90,000 tons of wet sludge might have to be removed annually from the tanks.

CONTACT BEDS.—As to the so-called contact beds or filters of broken stone, through which the effluent from septic tanks must be passed, it is our opinion, confirmed by Mr. Clark:

That the greatest permanent rate of filtration through contact beds of the best construction, meaning by this, contact beds of the material giving the best results and rates that can be maintained for a number of years, is not more than 800,000 gallons per acre per day, and perhaps not over 600,000 gallons per day.

The above rates refer to a "single contact" treatment of the effluent in the septic tanks, in beds of from 3 to 4 ft. in depth. If beds of double thickness were provided, say about 7 ft., it seems for present purposes to be feasible to increase the rate to about 1,000,000 gallons per acre daily. This is in accord with recent English experience, when we recognize the fact that the effluent of septic tanks in England contains much more organic and suspended matter than there would be in the case under consideration.

It will be necessary, therefore, to construct at the first works capable of purifying 100,000,000 gallons a day, after being pumped:

(1) Settling tanks for the deposit of the grosser mineral impurities held in suspension, from which the effluent will flow into,

(2) Septic tanks of at least 50,000,000 gallons capacity and 8 ft. deep, arranged to empty upon,

(3) Contact beds of broken stone of perhaps 167 acres in area, underdrained at the level of the highest tides.

COST OF MAINTENANCE.—As regards the cost of maintenance, there is uncertainty as to the kind and quality of water which would accumulate in the contact beds. In order for these beds to perform their function, it is necessary that the stones composing them should be covered with a slime, which requires considerable time to form and which accumulates to such an extent in the course of time as to diminish materially the capacity of the beds. The rate of this accumulation varies with the character of the sewage; it has been observed at some English cities that the accumulation was at a rate to diminish the capacity of the bed 50% in one year, while in other cases the diminution has only been 33% in three years.

When the capacity of the bed is materially diminished, it is necessary to remove the accumulation by washing the bed with water. The frequency of such washing, its cost, and the extent to which it should be carried are as yet indeterminate for contact beds, judging from their actual service, which in one case has been as long as eight years, but not under comparable conditions.

LIKELIHOOD OF OFFENSIVE ODORS.—It is necessary in the operation of a septic tank that there should be some sludge on the bottom and also a coating of sludge on the top. Where the surface is exposed to the wind, the formation of this surface sludge proceeds very slowly, and instances have occurred where a well formed surface has been suddenly dissipated by a gale of wind, and the process of formation has had to be repeated; the uncovered sewage may in the meantime give forth offensive odors.

It is not practical, therefore, at this time to make any close estimate of the cost of maintenance of the septic tank system applied to the purification of 100,000,000 or 200,000,000 gallons per day with any assurance that the cost in practice would correspond with the estimate.

The cost of the disposal plant necessary for the purification of 100,000,000 gallons per day, which is as much as should be provided for in the first disposal works to be built, may be approximately estimated and is about $4,500,000, including 300 acres of land, uncovered septic tanks, contact beds and two sludge boats to carry the sludge to sea. Covered tanks would correspondingly increase this sum.

COST OF THE PLAN RECOMMENDED.

The cost of construction of an outfall sewer to discharge 100,000,000 gallons daily from the pumping station to deep water near the center of New York Bay, would be about $1,500,000, with practically no operating charges. The great economy of the latter project is, therefore, apparent, and we have no hesitation in recommending it to you for adoption. It is not only much the least expensive of the possible solutions of the problem before you, but it is a final solution.

The estimate of cost of the sewers for the daily collection of 270,000,000 gallons of foul water from the Passaic watershed and of its discharge into New York Bay, as hereinbefore outlined, is as follows:

82,900 ft. of sewers from 7 ft. to 11.5 ft. diameter	$3,950,000
Pumping station at Wallington	50,000
Pumping station at Newark Bay	500,000
Outfall pipe for discharging 100,000,000 gallons daily from the west shore of Newark Bay to deep water in New York Bay	2,500,000
Total cost excluding cost of land	$7,000,000

THE EXCESSIVE COST OF SEPARATE DISPOSAL FOR PATERSON AND PASSAIC.

In considering the proposal for a trunk sewer to and purification work on the Hackensack meadows for Paterson, Passaic and other places in the upper district it was assumed that the works would provide for the ultimate disposal of 100,000,000 gallons a day, but the cost of operation was based on the treatment of 50,000,000 gallons a day, at the start. On this basis the plan would cost as follows:

Purification plant	$2,500,000
Special sewer from Paterson disposal line to works	829,000
Total	$3,329,000
Annual depreciation of plant	$42,500
Annual treatment	36,500
	$79,000
Capitalized at 4%	1,975,500
Total	$5,304,500

This sum is in excess of the assumed share for this section of the joint works for the whole valley.

In addition, the engineers believe that the proposed location of the plant "would eventually, if not at present, become objectionable to that part of the population which would eventually extend close to the meadows."

PROPOSED RECLAMATION OF THE ZUIDER ZEE.

Previous to 1892, a commission was appointed by the government of the Netherlands to investigate and report upon a project for reclaiming land now covered by the Zuider Zee. The scheme recently presented to the Dutch government is largely based upon the report of this commission.

It is sufficient to say here that the Zuider Zee, now forming an arm of the North Sea, was created in 1170; when a very heavy storm broke through the sand hills of the coast, which had before that protected the low-lying lands from inundation. The land bordering the River Yessel was thus converted into an inland sea 60 miles long; leaving the Texel and four other islands along the former coast line, and forming the higher portions of the flooded territory into the islands of Wieringen, Urk, Schokland and Marken.

The original idea of reclaiming the whole territory covered by the Zee—by uniting the coast islands, was abandoned as commercially impracticable. Instead of this the commission recommended the plan here shown. It is proposed to enclose the inner portion of the Zee by building an embankment—18 miles long and 18 ft. high—across from Wierengen to Piaam, on the Friesland shore; and to use this embankment for railway communication between North Holland and Friesland, and to thus shorten the distance between Amsterdam and Leeuwarden by 35 miles. Four other enclosing banks were then to be constructed, and the areas enclosed to be pumped out. An inland lake, 560 sq. miles in area, would be left for commercial purposes and acting as a reservoir for the waters of the Yessel and other waterways now emptying into the Zuider Zee. The overflow from this reservoir would discharge into the North Sea through 30 sluices formed in the Wierengen-Piaam embankment.

The advantages claimed for this scheme are as follows: As the level of the lake will be lower than that of the sea, it will better drain the low-lying land; and as the lift will be less, the cost of pumping will be decreased. The length of sea and coast wall is reduced to a minimum by the outer embankment; and this embankment shortens distances by railways. To provide for navigation with the water level lowered and tidal influence eliminated, access to harbors must be secured by dredging the approaches. A ship-canal with a bottom width of 65.5 ft. is also projected; to run from Harlingen, along the Friesland dike to Piaam with a terminal lock 26.5 ft. wide and 19 ft. long. As the fishing industry would be destroyed by the exclusion of sea water, the fishermen must be compensated for this loss.

The estimated cost of the project to be first carried out is $39,583,335; and this contemplates the building of the western embankments only, Nos. 1, 2. For the completed plan the estimate amounts to $78,750,000. The cost of the great sea-dike is set down at $11,720,835. This money would be raised by a loan, repayable in 60 years; and it is expected that it would require nine years to build the sea-dike; 18 years to enclose the areas 1 and 2, and 33 years to complete the project.

Map Showing Land to be Reclaimed from the Zuider Zee, Holland.

The area of fertile land in the two "polders" 1 and 2 is 115,000 acres; with 365,000 acres in the other "polders" 3 and 4. The net renting value of the land to be reclaimed is estimated at $10 per acre; or, $1,150,000 for the west side and $3,650,000 for the east side. The bill presented claims that under the present project there would be a revenue of $316,665 per year, after paying interest and principal.

In Engineering News for Sept. 15, 1892, an abstract is given of the report of the old commission; and the map here presented is taken from that issue, and is essentially the plan now proposed. It is there stated that the great sea-dike, connecting Wierengen and Piaam, would be built in water 13 to 20 ft. deep. It was then proposed to found this dam on mattress work, on a base 525 ft. wide. The outer or North Sea face would be protected by fascines to the level of low water and the slope above that covered with stone. The material for the dam was to be obtained from the excavation of a ship-canal across the Island of Wierengen; though the present plan seems to contemplate only one exit for vessels, on the Piaam side.

U. S. Consul Hill, of Amsterdam, under date of Sept. 23, 1901, stated that this project has been withdrawn from the States-General by the new ministry, and is thus disposed of for a considerable time. This is made necessary by the present condition of the Dutch finances.